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U. S. Army Engineer Waterways Experiment Station

Research in Earth Physics

Phase Report No. 1
Part II

THE INFLUENCE OF STRESS SYSTEM ON THE BEHAVIOR OF SATURATED CLAYS DURING UNDRAINED SHEAR

Charles C. Ladd
Julius Varallyay

Research Report R65-11
Soils Publication No. 177

July, 1965

MIT

DEPARTMENT OF CIVIL ENGINEERING

SCHOOL OF ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY
Cambridge 39, Massachusetts

THE INFLUENCE OF STRESS SYSTEM ON THE BEHAVIOR OF SATURATED CLAYS DURING UNDRAINED SHEAR

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Vicksburg, Mississippi

Soil Mechanics Division
Department of Civil Engineering
Massachusetts Institute of Technology

Research Report R65-11

ABSTRACT

Stability and deformation problems involving the undrained shear of deposits of saturated clay require determination of one or more of the following parameters: the in situ undrained shear strengths (s_u); the effective stress parameters defining the Mohr-Coulomb failure envelope ($\bar{c}$, $\bar{\Phi}$); Skempton's pore pressure parameter (A); and a stress-strain relationship (modulus E or a stress-strain curve). An accurate prediction of these parameters from the results of field and/or laboratory shear tests requires:

1. Testing of samples which have the same properties as the in situ clay;
2. Performance of shear tests which have the same stress system, rate of strain and environment as will be imposed in the field, i.e., measurement of the correct soil parameters.

This study investigates one phase of the overall problem of determining in situ properties, namely, the effects of stress system variables on the undrained shear behavior of saturated clays. Stress system variables refer to the direction and relative magnitude of the three principal stresses during consolidation and during shear. The report reviews and analyzes previous work in the area and presents the results of an extensive series of consolidated-undrained triaxial tests with pore pressure measurements on normally consolidated Boston blue clay prepared from a dilute slurry.

The effects on undrained shear behavior of the following topics are considered in detail: anisotropic consolidation, perfect sampling, the intermediate principal stress, and rotation of principal planes during shear. These variables are shown to have a significant influence on most of the strength parameters and such

effects should be taken into account in important stability and deformation problems.

The $\phi = 0$ (total stress) method of stability analysis commonly assumes an unique in situ undrained shear strength. Data in the report show that this will not generally be true because the in situ mode of failure (i.e., stress system) can have a pronounced effect on undrained shear strength. For normally consolidated clay deposits, the in situ strength for a strutted excavation or an embankment can be far less than that for a vertical cut or that obtained from an unconfined compression test on a "perfect sample."

The reported success of the $\phi = 0$ analysis is questioned because:

1. The methods commonly used to determine s_u, such as the field vane, the unconfined compression test, and the consolidated-undrained triaxial test, seldom yield consistent results;

2. The above methods rely upon compensating errors for their success in many instances.

Section 5.3 of the report presents a detailed illustration of the problems associated with a $\phi = 0$ analysis for several types of field cases. It is emphasized that the analyses involving important structures should not rely solely on the results of unconfined and/or field vane tests.

Additional research on means for coping with sample disturbance and on the influence of the in situ stress system is required before the engineer can select with confidence strength parameters for undrained shear. In particular, laboratory shear testing programs should consider the value of K at consolidation, the intermediate principal stress at failure, and the direction of the major principal stress at failure relative to its direction after consolidation (i.e., rotation of principal planes).

FOREWARD

The work described in this report was performed under Contract No. DA-22-079-eng-330 entitled "Research Studies in the Field of Earth Physics" between the U.S. Army Engineer Waterways Experiment Station and the Massachusetts Institute of Technology. The research is cosponsored by the U.S. Army Materiel Command under DA Projects 1-V-0-14501-B-52A-30, "Earth Physics (Terrain Analysis)," and 1-V-0-21701-A-046-05, "Mobility Engineering Support," and by the Directorate of Remote Area Conflict, Advanced Research Projects Agency, under the "Mobility Environmental Research Study," ARPA Order No. 400.

The general objective of the Research in Earth Physics is the development of a fundamental understanding of the behavior of particulate systems, especially cohesive soils, under varying conditions of stress and environment. Work on the project, initiated in May 1962, has been carried out in the Soil Mechanics Division (headed by Dr. T. William Lambe, Professor of Civil Engineering) of the Department of Civil Engineering under the supervision of Dr. Charles C. Ladd, Associate Professor of Civil Engineering.

This report presents only one portion of the overall research being conducted under the contract. Phases currently under investigation are:

1. In Situ Strength and Compression Properties of Natural Clays.

   a. Effects of sample disturbance (i.e., excessive shear strains) on the undrained strength, stress-strain modulus, and one-dimensional compression behavior of natural clays.

b. Effects of stress-system variables (anisotropic consolidation, intermediate principal stress, rotation of principal planes) on stress-strain behavior of clays during undrained shear.

2. Influence of Environment on Strength and Compression Properties of Soils.

   a. Effect of high vacuum and temperature on the properties of granular systems.

   b. Effects of natural cementation and type of pore fluid on the strength and compression properties of saturated clays.

   c. The strength of clays at very low effective stresses and especially the nature and magnitude of "true cohesion."

3. The Structure of Clay.

   a. Nature and magnitude of interparticle forces in clay-water systems.

   b. Fabric of kaolinite

Many of the above topics complement and/or draw information from other research projects in the Soil Mechanics Division. These include support from the Office of Naval Research and The National Science Foundation (Grant G-19440).

This report was written by Professor Ladd with the assistance of Mr. Julius Varallyay, former Research Assistant in the Soil Mechanics Division. Mr. Varallyay performed the experimental work presented in Chapters 3 and 4. Mr. Paulo da Cruz, former Research Assistant, and Mr. William A. Bailey, Research Assistant, ran the triaxial tests on the Vicksburg Buckshot and Kawasaki clays reported in Chapter 2.

This report is Part II of Phase Report No. 1. Part I, entitled "Stress-Strain Behavior of Saturated Clay and Basic Strength

Principles" by C. C. Ladd, was submitted in April 1964. It presented a simplified picture of the strength behavior of clays for use as a framework with which to study the properties of actual clays in terms of deviations from this idealized picture. In essence, Part I presented the background material required for the presentation, analysis, and comprehension of the experimental data and conclusions presented herein.

Pertinent reports issued under this research contract are:

1. "Research in Earth Physics, Progress Report for the period June 1962 - December 1962," Department of Civil Engineering Publication R63-9, M.I.T., Feb. 1963.

2. Ladd, C. C., "Stress-Strain Behavior of Saturated Clay and Basic Strength Principles," Phase Report No. 1, Part 1, Department of Civil Engineering Publication R64-17, M.I.T., April 1964.

3. Bromwell, L. G., "Adsorption and Friction Behavior of Minerals in Vacuum," Phase Report No. 2, Department of Civil Engineering Publication R64-42, M.I.T., March 1965. (In press).

4. Bailey, W. A., "The Effects of Salt on the Consolidation Behavior of Saturated Remolded Clays," Phase Report No. 3, Department of Civil Engineering Publication R65-19, M.I.T., May 1965. (Submitted for review in May 1965).

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CHAPTER 1

INTRODUCTION*

1.1 TYPES OF PARAMETERS AND TEST METHODS FOR STABILITY AND DEFORMATION ANALYSES FOR UNDRAINED SHEAR

Among the most difficult problems facing the civil engineer are those involving the stability and deformation of deposits of saturated clay. Examples include the bearing capacity and settlement of footings, trafficability, the stability of cut slopes, stress distribution in layered deposits and the factor of safety of excavations against bottom upheaval. Realistic predictions of field behavior are often difficult on two counts: lack of an appropriate method of analysis; and the problem of selecting the appropriate soil parameters to plug into the theoretical analyses. For example, the theory of elasticity is used for the solution of many stress distribution and soil deformation problems even though the soil engineer knows that soil is not an isotropic, linear-elastic material. Moreover, the difficulties in selecting an "elastic modulus" for these computations are formidable (Ladd, 1964).

An oil tank constructed on a deposit of soft saturated clay (Fig. 1.1) is used to illustrate the types of soil parameters and test methods which might be employed in analyses for stability and deformation. If the tank is filled rapidly, so that no water drains from the clay deposit, analyses and parameters of interest to the civil engineer include:

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* Appendix A lists notations used throughout the report.

1. The factor of safety (F.S.) against rupture using a "$\phi = 0$" analysis (Skempton, 1948), which requires a determination of the undrained shear strength, s_u, which existed in situ prior to filling of the tank;

2. An estimate of the immediate settlement (due to strains during undrained shear), requiring a knowledge of a stress-strain modulus, E.*

If the above analyses indicate instability and/or excessive settlements, the tank might be filled in stages in order to allow for the partial consolidation and an increase in the undrained shear strength. In this case, the engineer might want to know:

3. The relationship between consolidation pressure and undrained shear strength in order to perform a total stress stability analysis;

4. The effective stress parameters $\bar{c}$ and $\bar{\phi}$ defining the Mohr-Coulomb failure envelope and Skempton's (1954) pore pressure parameter A in order to perform an effective stress stability analysis utilizing values of pore water pressure measured in the field. (Bishop and Bjerrum, 1960, present an excellent discussion on the use of effective stress stability analyses and its relationship to total stress analyses; Lambe, 1962a, has discussed some of the problems of predicting and interpreting pore pressures in the field.)

In summary, those soil parameters of interest are:

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* One might also employ Lambe's (1964) stress path method which uses strains measured in CU triaxial tests subjected to the in situ stress increments (computed from the theory of elasticity).

s_u = undrained shear strength, both prior to and during filling,

$\bar{c}$ and $\bar{\Phi}$ = cohesion intercept and friction angle defining the failure envelope for undrained shear,

A = pore pressure parameter for undrained shear,

E = stress-strain modulus for undrained shear.

The test methods used to obtain these parameters are varied; a partial listing of some of the more common methods is given in Table 1.1. Unfortunately, the different methods that are employed to find a given parameter often yield conflicting results. Examples of this are illustrated below.*

The three most common methods of estimating the in situ s_u are field vane tests, unconfined compression (or triaxial UU) tests on "undisturbed" samples, and consolidated-undrained (CU) triaxial tests on "undisturbed" samples where the specimens are consolidated with the in situ stresses. An analysis of numerous cases from all over the world showed the following:

1. For 20 cases outside Norway** on all types of clays, comparing field vane to unconfined compression and triaxial UU on typical tube samples:

$$\frac{s_u \text{ (U and UU)}}{s_u \text{ (field vane)}} = 0.7 \text{ (0.4 to 1.0)}$$

---

* Bishop and Bjerrum (1960) emphasize those cases where test methods are consistent and apparently yield good estimates of field behavior.

** Vold (1956) reports that unconfined compression tests yield slightly higher strengths, on the average, than the field vane for normally consolidated and slightly overconsolidated clay deposits in Norway.

2. For 11 cases from throughout the world (Table 2 of Ladd and Lambe, 1963), comparing unconfined compression and triaxial UU to consolidated-undrained triaxial tests on specimens isotropically consolidated to the overburden pressure (CIU tests with $\bar{\sigma}_c = \bar{\sigma}_{vo}$), both run on tube samples:

$$\frac{s_u\,(\text{U and UU})}{s_u\,(\text{CIU, } \bar{\sigma}_c = \bar{\sigma}_{vo})} \approx 0.66\ (0.4 \text{ to } 1.0)$$

A most striking example of possible discrepancies is found in strength data obtained on the Leda clay from Ottawa, Canada. For clay at a depth of 55 to 60 feet, Coates and McRostie (1963) report:

Type of Test and Sample	s_u (tons/ft^2)
1. Field vane	0.85
2. Unconfined compression and triaxial UU	
a. 2 in. dia. open drive	0.6
b. 3.4 in. dia. fixed piston	1.1
c. block sample	1.6
3. CIU triaxial consolidated to overburden pressure	
a. 2 in. dia. fixed piston	0.9
b. N.G.I. piston sampler	1.35
c. block sample	1.65

The clay is overconsolidated, moderately plastic, and very sensitive with a high liquidity index. The strengths varied from 0.6 to 1.65 depending upon the type of test and the type of sample. However, engineering practice often assumes that any one of these methods would yield the in situ strength. Some of the reasons for the wide range in measured strengths are discussed in the next section.

The other strength parameters are also subject to wide variations depending upon the methods used to obtain them. As examples, Bjerrum and Simons (1960) discuss the factors influencing $\bar{\phi}$, A_f and $s_u/\bar{\sigma}_{vo}$ for normally consolidated clays; Lambe (1962a) illustrates the problems in measuring the pore pressure parameter A; and Ladd (1964) shows the large effects that type of test have on measured values of the stress-strain modulus E.

1.2 FACTORS INFLUENCING STRENGTH PARAMETERS FOR UNDRAINED SHEAR

The general requirements for accurately measuring the strength parameters of a natural clay during shear are:

1. Performing tests on specimens having the same "soil structure" (Lambe, 1958), and hence engineering properties, as the in situ clay;

2. Performing tests on specimens in a manner to ensure that the stress system, time, and environment (temperature, pore fluid characteristics, etc.) are the same as will be imposed in the field.

Some of these requirements are spelled out in more detail in Table 1.2, which lists the factors influencing strength parameters measured with undrained triaxial tests on samples of clay.

Examples of how the common types of shear tests fail to meet the basic requirements of simulating in situ strength behavior are:

1. The field vane may test a specimen having the in situ water content, preshear stress system, and environment (and hence soil structure) but the stress system commensurate with a vertical, cylindrical failure plane and the rapid strain rate hardly duplicate the mode of failure and rate of shear usually found in the field;

2. The unconfined compression test would usually have the in situ water content and an anisotropically

consolidated triaxial compression test might duplicate the in situ preshear stress system, but these tests have little else in common with a clay element sheared in the field.

3. An elaborate plane strain shear device might duplicate an actual stress system in the field, but any disturbance in getting a sample from the ground into the laboratory equipment would preclude testing a sample having the same properties as the clay in the field.

It is obviously impossible to exactly reproduce field behavior in a laboratory test. On the other hand, it is not necessary to duplicate field behavior in every respect in order to arrive at parameters to use in most engineering analyses. There is a question, however, as to which of the many field conditions must be duplicated, at least approximately, in order to obtain reasonably accurate parameters. Current knowledge is wholly deficient regarding the most important variables, the errors to be expected, and the steps which can be taken in order to arrive at reasonable answers.

Table 1.2 lists the major factors (sample disturbance, stress system, time and environment) effecting strength behavior. Sample disturbance has lately received increased attention (Ladd and Lambe (1963), Ladd (1964), and Seed, Noorany and Smith, 1964) regarding its affects on values of s_u and E and possible means of correcting for it, but much is yet unknown. The influence of time has been studied extensively; for example, Casagrande and Wilson (1951), Bjerrum, et al (1958), Crawford (1959), and Richardson and Whitman (1963) on strain rate effects; Moretto (1948) and Mitchell (1960) on thixotropy; and Ladd (1961), Wissa (1961), and Bjerrum and Lo (1963) on effects of aging. The importance of environmental effects have been illustrated by: Ladd (1961), Mitchell and Campanella (1963, and Mitchell (1964) on the effects of temperature changes; Samuels (1950) Bjerrum and Rosenqvist (1956), Leonards and Andersland (1960), Ladd (1961), Bailey (1961), Wissa (1961), and Olson (1963) on the effects of salt concentration and/or cation valency.

The influence of stress system, at least with saturated clays, has probably received the least amount of attention. The next two sections illustrate the different types of stress systems and present the scope of the experimental program on the effects of stress systems on the undrained strength behavior of normally consolidated Boston blue clay.

1.3 STRESS SYSTEM VARIABLES (see Part I of this report for additional background information)

Stress system includes both the stress system existing prior to shear and the stress system applied during shear. In turn, stress system means the direction and relative magnitude of the three principal stresses.

The stress system prior to shear is that resulting from the consolidation stresses. The two most common types of stress conditions obtained in the laboratory are isotropic consolidation (equal principal stresses) and one-dimensional consolidation, such as in the standard oedometer test. In the latter test, the ratio of the horizontal to the vertical consolidation pressure* is called the coefficient of earth pressure at rest, i.e., $K_o = \bar{\sigma}_{hc}/\bar{\sigma}_{vc}$. For most normally consolidated clays, K_o equals 0.6 ± 0.2 and is approximately related empirically to the friction angle by $K_o = 1 - \sin \bar{\Phi}$. The value of K_o increases with rebound and becomes greater than unity at overconsolidation ratios exceeding about 3.5 ± 1. The variation in K_o with overconsolidation ratio for three clays is shown in Fig. 1.2.**

The three basic types of stress systems that can be applied

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* One should correctly use consolidation stress rather than consolidation pressure.

** Brooker and Ireland (1965) present an excellent article on the influence of soil type and stress history on the value of K_o.

during shear, which depend upon the relative magnitude of the applied intermediate principal stress $\Delta\sigma_2$, are presented in Fig. 1.3. These are: 1) triaxial compression where $\Delta\sigma_2 = \Delta\sigma_3$; 2) triaxial extension where $\Delta\sigma_2 = \Delta\sigma_1$; and 3) plane strain where $\Delta\sigma_2$ is intermediate between $\Delta\sigma_1$ and $\Delta\sigma_3$ and where all strains in the soil are parallel to the plane of $\Delta\sigma_1$ and $\Delta\sigma_3$.

Stress systems typically encountered in the field are illustrated in Fig. 1.4 for a normally consolidated clay with K_o stresses so that σ_1 initially acts in the vertical direction and $\sigma_2 = \sigma_3$ acts in the horizontal direction. Let us look at what happens during undrained shear.

Case (a). Under center line of a circular footing:

The vertical stress increases more than the horizontal stresses increase and the directions of the principal stresses remain unchanged. The applied stress system is that of triaxial compression ($\sigma_2 = \sigma_3$).

Case (b). Under center line of a circular excavation:

The vertical stress decreases more than the horizontal stresses decrease so that the horizontal stresses could eventually exceed the vertical stress. If such occurred, the soil would be in a state of triaxial extension ($\sigma_2 = \sigma_1$). Moreover, there would be a rotation of principal planes since the major principal stress now acts in the horizontal direction.

Case (c). Under center line of a strip footing:

This case is similar to that of Case (a), except that the increase in the longitudinal stress ($\Delta\sigma_x$) is larger than the increase in the tranverse stress ($\Delta\sigma_y$). The soil is in a state of plane strain with the major principal stress still acting in the vertical direction. As failure is approached, the intermediate principal stress(longitudinal stress) would be approximately equal to the average of the other two principal stresses (Henkel, 1960b).

Case (d). Behind a retaining wall with a passive pressure:

This is another case of plane strain, but with a rotation of the principal planes since the major principal stress becomes equal to the tranverse stress.

If the clay in Fig. 1.4 had been heavily overconsolidated with K_o greater than unity, Cases (a) and (c) would have exhibited a rotation of principal planes since σ_1 would have acted in the horizontal direction prior to shear and in the vertical direction at failure. Conversely, σ_1 would always act in a horizontal direction in Cases (b) and (d).

The purpose of this report is to show how the stress system, both at consolidation and that applied during shear, influences the undrained strength behavior of saturated clay. Data will show, for example, that the undrained strength of a normally consolidated clay element under the center line of a circular footing (Case (a), Fig. 1.4) may be two to three times larger than the values of s_u for a clay element with identical consolidation stresses but sheared under the center line of a circular excavation (Case (b), Fig. 1.4).

The above behavior is illustrated by the hypothetical stress paths presented in Fig. 1.5. Clay has been normally consolidated with $K_o = 0.5$ to point A' ($\bar{\sigma}_{vc} = 1.50$, $\bar{\sigma}_{hc} = 0.75$). There is a static pore pressure of 0.50, so that the total stresses are represented by point A ($\sigma_{vc} = 2.00$, $\sigma_{hc} = 1.25$). The clay element sheared undrained to failure under the footing has a total stress path AB; the effective stresses $\bar{\sigma}_v$ and $\bar{\sigma}_h$ are shown by the path $A'B'$. At failure, $(\sigma_v - \sigma_h)_f = BE = (\bar{\sigma}_v - \bar{\sigma}_h)_f = B'E' = (\sigma_1 - \sigma_3)_f = 1.05$.

The clay element sheared undrained to failure under the excavation has a total stress path AC and an effective stress path $A'C'$. When these paths cross the $K = 1$ line, the horizontal stress is larger than the vertical stress and the directions of the major and minor principal planes have rotated by 90°. At failure $(\sigma_h - \sigma_v)_f = CF = (\bar{\sigma}_h - \bar{\sigma}_v)_f = C'F' = (\sigma_1 - \sigma_3)_f = 0.40$. The ratio of undrained strengths is therefore equal to 1.05/0.40 = 2.62.

1.4 SCOPE OF EXPERIMENTAL INVESTIGATION

A series of consolidated-undrained triaxial tests with pore pressure measurements ($\overline{CU}$ tests) are run on normally consolidated samples of saturated Boston blue clay. The principal variables are.*

1. Value of K
   a. $K = 1$ ($\bar{\sigma}_{ac} = \bar{\sigma}_{rc}$)
   b. $K = K_o$ ($\bar{\sigma}_{ac} > \bar{\sigma}_{rc}$)
   c. $K = 1/K_o$ ($\bar{\sigma}_{ac} < \bar{\sigma}_{rc}$)
2. Value of σ_2 at failure
   a. $\sigma_2 = \sigma_3 = \sigma_r$ (failure in compression)
   b. $\sigma_2 = \sigma_1 = \sigma_r$ (failure in extension)
3. Effect of perfect sampling (release of K_o stresses followed by failure in compression)

Secondary variables are:

1. Value of major principal consolidation pressure $\bar{\sigma}_{1c}$
2. Stress controlled versus strain controlled undrained shear.

Figure 1.6 shows the different consolidation stresses and total stress paths used in the investigation, for a given value of $\bar{\sigma}_{1c}$. The initial portion of the paths has been drawn at an angle of 45^o (i.e., $\Delta\sigma_a = -\Delta\sigma_r$ during undrained shear) in order to illustrate the general direction, and does not represent the actual path.** The types of tests are:

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* σ_a and σ_r refer to axial and radial stresses.

** Section II B 2 of Part I of this rep has already explained that for a given value of ($\Delta\sigma_1 - \Delta\sigma_3$), the actual magnitude of the change in the smaller of the two stresses applied during shear has no influence on effective stress behavior as long as Skempton's B parameter is equal to unity.

1. $\overline{\text{CIU}}$ C: compression test on isotropically consolidated sample.

2. $\overline{\text{CIU}}$ E. extension test on isotropically consolidated sample.

3. $\overline{\text{CK}_o\text{U}}$ C: compression test on K_o consolidated sample.

4. $\overline{\text{CK}_o\text{U}}$ RE: extension test on K_o consolidated sample (σ_r increased and/or σ_a is decreased until failure is reached).

5. $\overline{\text{C(1/K}_o\text{)U}}$ E: extension test on $1/K_o$ consolidated (i.e., $\bar{\sigma}_{rc}$ is greater than $\bar{\sigma}_{ac}$) sample.

6. $\overline{\text{C(1/K}_o\text{)U}}$ RC: compression test on $1/K_o$ consolidated sample (σ_a is increased and/or σ_r is decreased until failure is reached).

7. $\text{C(K}_o\text{)}$ -$\overline{\text{UU}}$: compression test on "perfect" sample after K_o consolidation. Perfect sampling denotes an undrained release of K_o stresses to attain an isotropic state of stress (Ladd and Lambe, 1963).

Some of the above tests duplicate possible field conditions ($\overline{\text{CK}_o\text{U}}$ C and $\overline{\text{CK}_o\text{U}}$ RE tests represents Cases a and b in Fig. 1.4) or triaxial UU compression tests on perfect samples (CK_o - $\overline{\text{UU}}$ tests). The $\overline{\text{CIU}}$ C test is the most common type of triaxial test run in the laboratory. The other tests were selected in order to investigate the effects of σ_2 on strength behavior, and represent the extreme case of extension stresses during consolidation and/or shear. It would have been preferable to run plane strain tests, where σ_2 is between σ_1 and σ_3, but equipment for such tests was not available.*

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* Tests employing plane strain and simple shear are planned for the future

It is thought, however, that the results of plane strain tests might lie between those from the compression and extension tests.

TABLE 1.1

DETERMINATION OF STRENGTH PARAMETERS
FOR UNDRAINED SHEAR OF SATURATED CLAY

Parameter	Type of Analysis	Methods of Determination	
In situ undrained shear strength, s_u	Total stress analysis for undrained shear (ϕ = 0 analysis)	Empirical	1. P.I. vs. $s_u/\bar{\sigma}_{vo} = c/p$
		Field	2. Field vane 3. Cone penetration 4. Split-spoon penetration
		Lab UU	5. Unconfined compression 6. Triaxial UU 7. Miniature vane 8. Cone penetration 9. Ring shear
		Lab CU	10. Triaxial CU 11. Direct shear CU 12. Simple shear CU
Effective stress envelope for undrained shear, $\bar{c}$, $\bar{\phi}$	Effective stress analysis for undrained and/or partially drained cases	Lab $\overline{CU}$	1. Triaxial $\overline{CU}$ 2. Direct shear $\overline{CU}$ 3. Simple shear $\overline{CU}$
		Lab $\overline{CD}$	4. Triaxial CD 5. Direct shear CD 6. Simple shear CD
Pore pressure parameter, A	Effective stress analysis for undrained shear or for settlement analyses	Lab $\overline{CU}$	1. Triaxial $\overline{CU}$
Stress-strain modulus, E		Empirical	1. $E = (200\text{-}400)\, s_u$
		Field	2. Plate bearing
		Lab UU	3. Unconfined compression 4. Triaxial UU
		Lab CU	5. Triaxial CU

TABLE 1.2

FACTORS INFLUENCING STRENGTH PARAMETERS FOR UNDRAINED SHEAR OF SATURATED CLAY

Type of Test	Parameters
Triaxial UU	s_u, E
Triaxial $\overline{CU}$	s_u, $\bar{c}$, $\bar{\phi}$, A, E

Factor	Variable	Type of Test
Sample Disturbance	Preshear:	
	1. Effective stress	1. UU, $\overline{CU}$
	2. Water content	2. UU, $\overline{CU}$
Stress System	Preshear:	
	1. Total stress level	1. UU
	2. Effective stress level	2. $\overline{CU}$
	3. Effective stress ratio, K_c	3. $\overline{CU}$
	During Shear:	
	4. Total stress level	4. UU, $\overline{CU}$
	5. Value of σ_2	5. UU, $\overline{CU}$
	6. Rotation of principal planes	6. $\overline{CU}$
	7. Cyclic loading	7. UU, $\overline{CU}$
Time	Preshear:	
	1. At constant water content (thixotropy)	1. UU
	2. At constant effective stress (aging)	2. $\overline{CU}$
	During Shear:	
	3. At constant water content (strain rate effects)	3. UU, $\overline{CU}$
Environment	Preshear and During Shear:	
	1. Temperature	1. UU, $\overline{CU}$
	2. Pore fluid composition	2. UU, $\overline{CU}$

Fig. I.1 Stability and Deformation of an Oil Tank on a Soft Saturated Clay Deposit

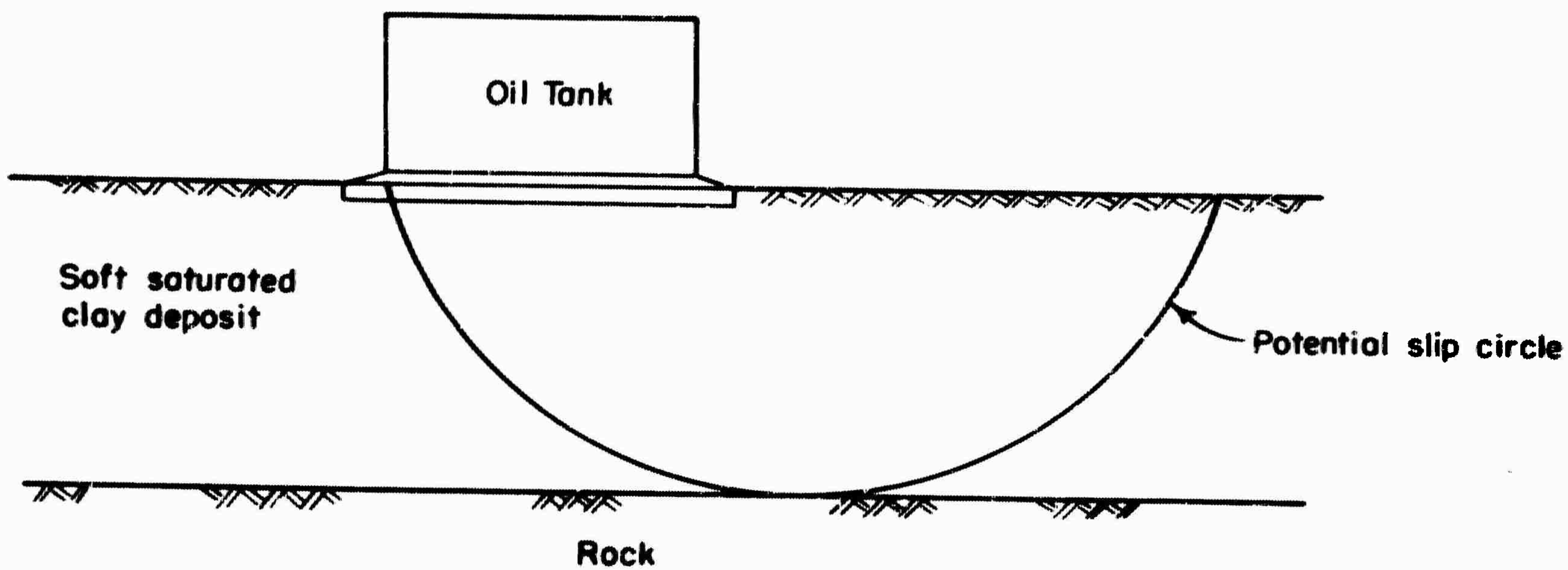


Fig. 1.2. Coefficient of Earth Pressure at Rest Versus Overconsolidation Ratio

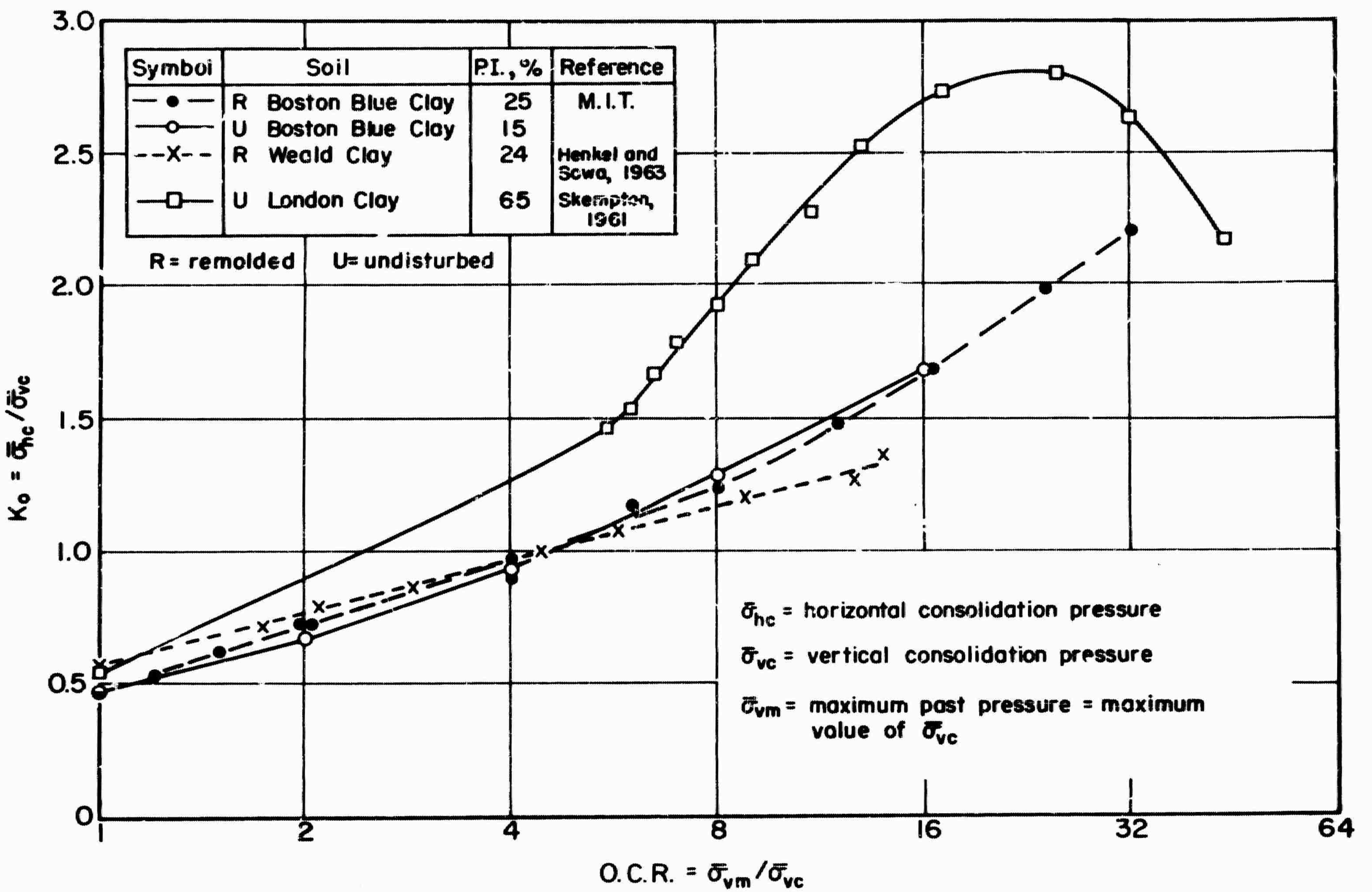
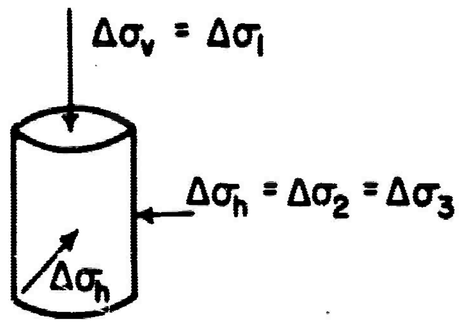


Fig. 1.3. Three Basic Types of Stress Systems

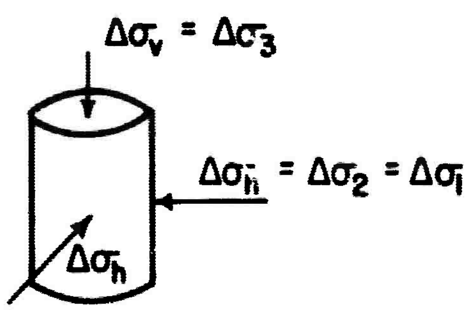
(a) Triaxial Compression $\Delta\sigma_1 > \Delta\sigma_2 = \Delta\sigma_3$



Loading:
$\Delta\sigma_v$ is positive
$\Delta\sigma_h = 0$

Unloading:
$\Delta\sigma_v = 0$
$\Delta\sigma_h$ is negative

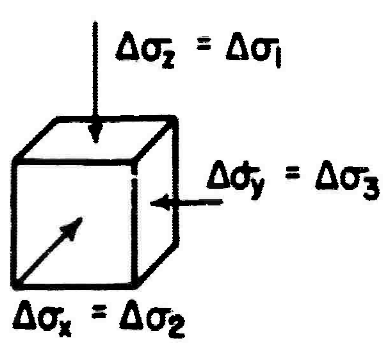
(b) Triaxial Extension $\Delta\sigma_1 = \Delta\sigma_2 > \Delta\sigma_3$



Loading:
$\Delta\sigma_h$ is positive
$\Delta\sigma_v = 0$

Unloading:
$\Delta\sigma_h = 0$
$\Delta\sigma_v$ is negative

(c) Plane Strain $\Delta\sigma_1 > \Delta\sigma_2 > \Delta\sigma_3$; all strains in plane of $\Delta\sigma_z$ and $\Delta\sigma_y$

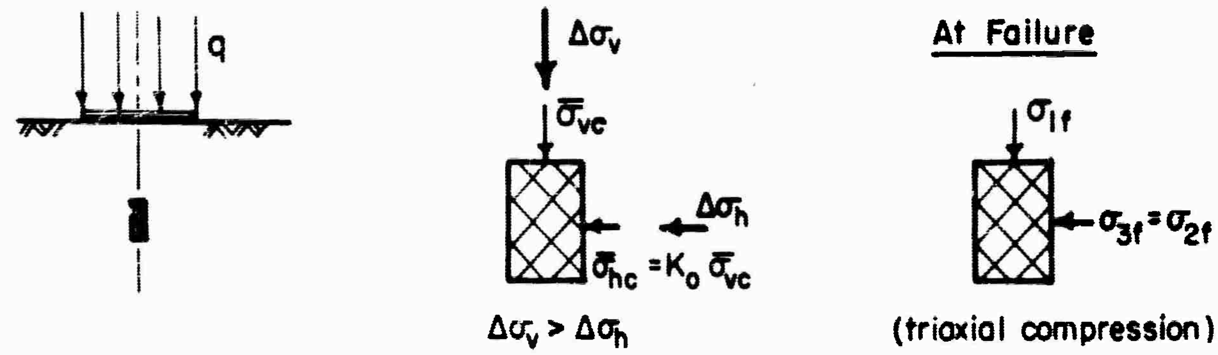


Loading:
$\Delta\sigma_z$ is positive
$\Delta\sigma_y = 0$

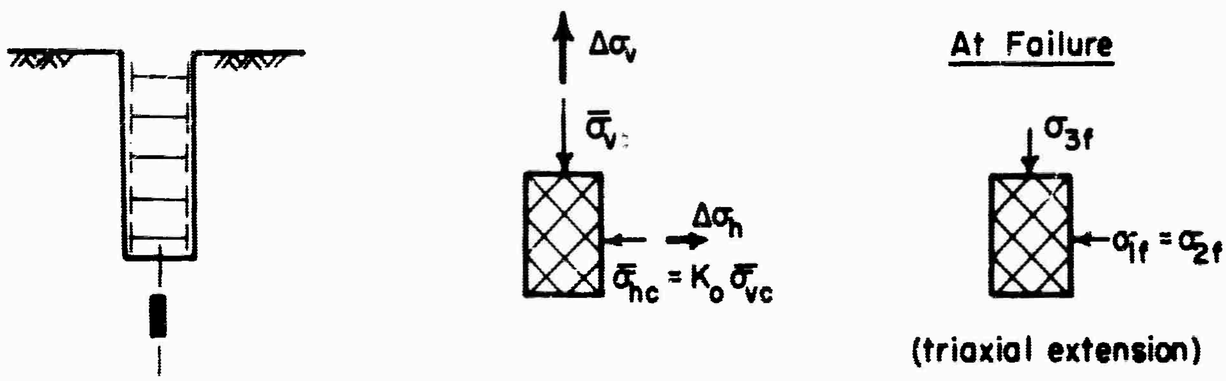
Unloading:
$\Delta\sigma_z = 0$
$\Delta\sigma_y$ is negative

Fig.1.4 Typical Stress Systems in the Field for a Normally Consolidated Clay (footings and walls are assumed to have frictionless surfaces)

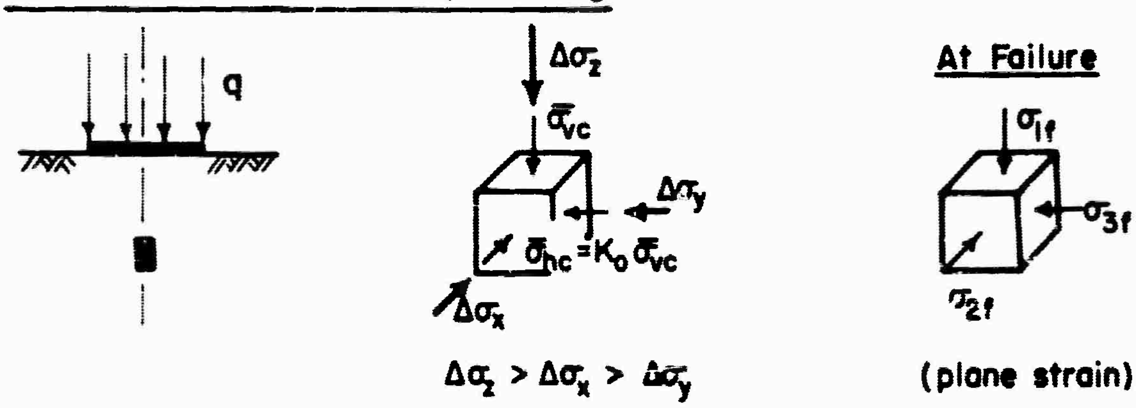
(a) Under Centerline of a Circular Footing



(b) Under Centerline of a Circular Excavation



(c) Under Centerline of a Strip Footing



(d) Retaining Wall with a Passive Pressure

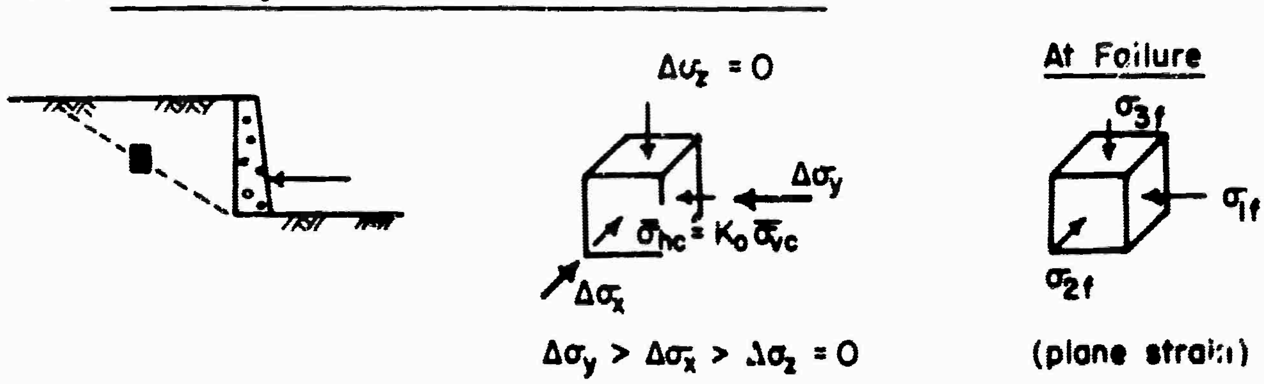


Fig. 1.5. Stress Paths for Clay Elements Under the Centerline of a Circular Footing and a Circular Excavation for Undrined Shear

Case	Stress Path		$(\sigma_1 - \sigma_3)_f$
	Total	Effective	
Footing	AB	A' B'	BE = B' E'
Excavation	AC	A' C'	CF = C' F'

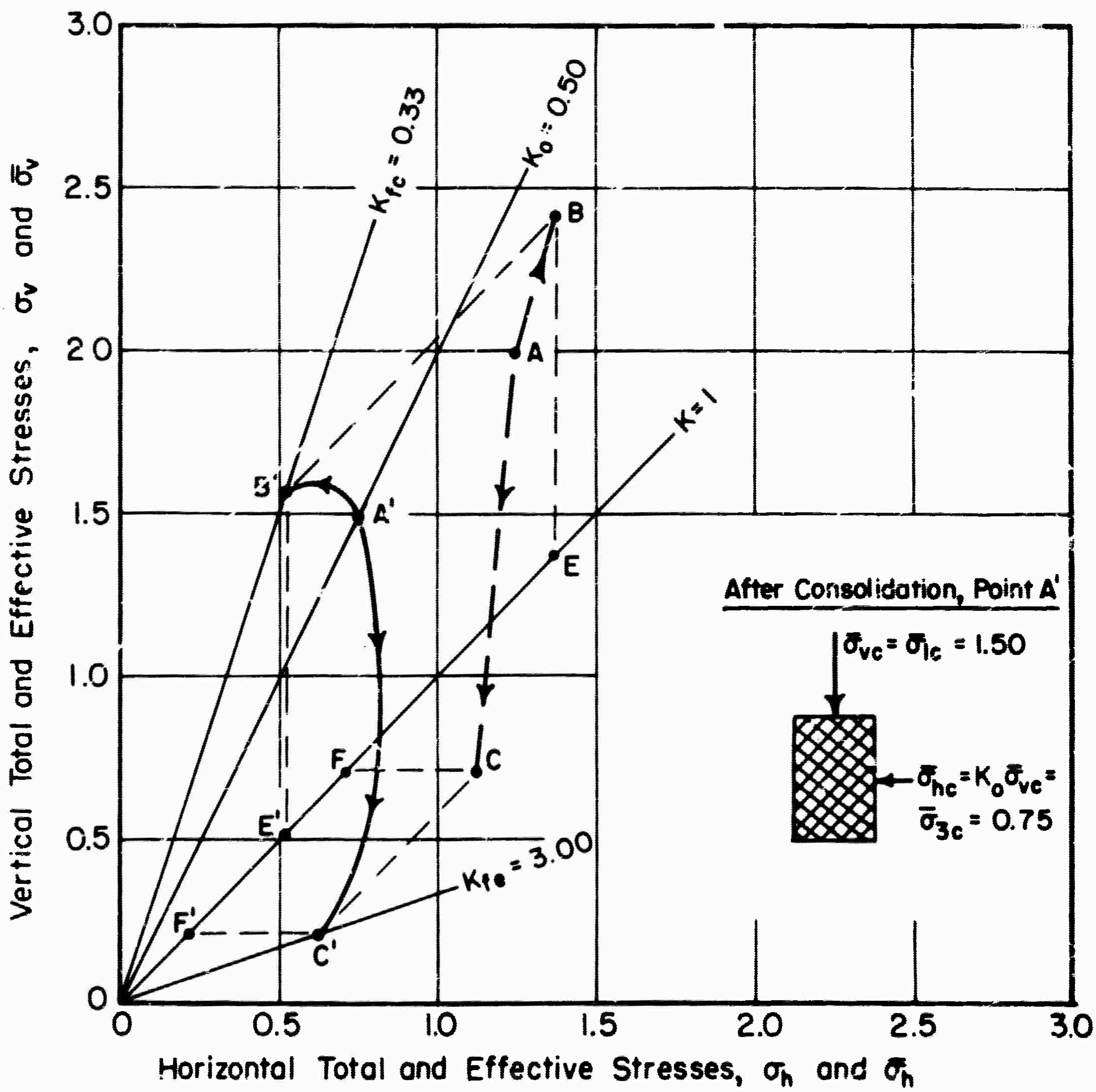
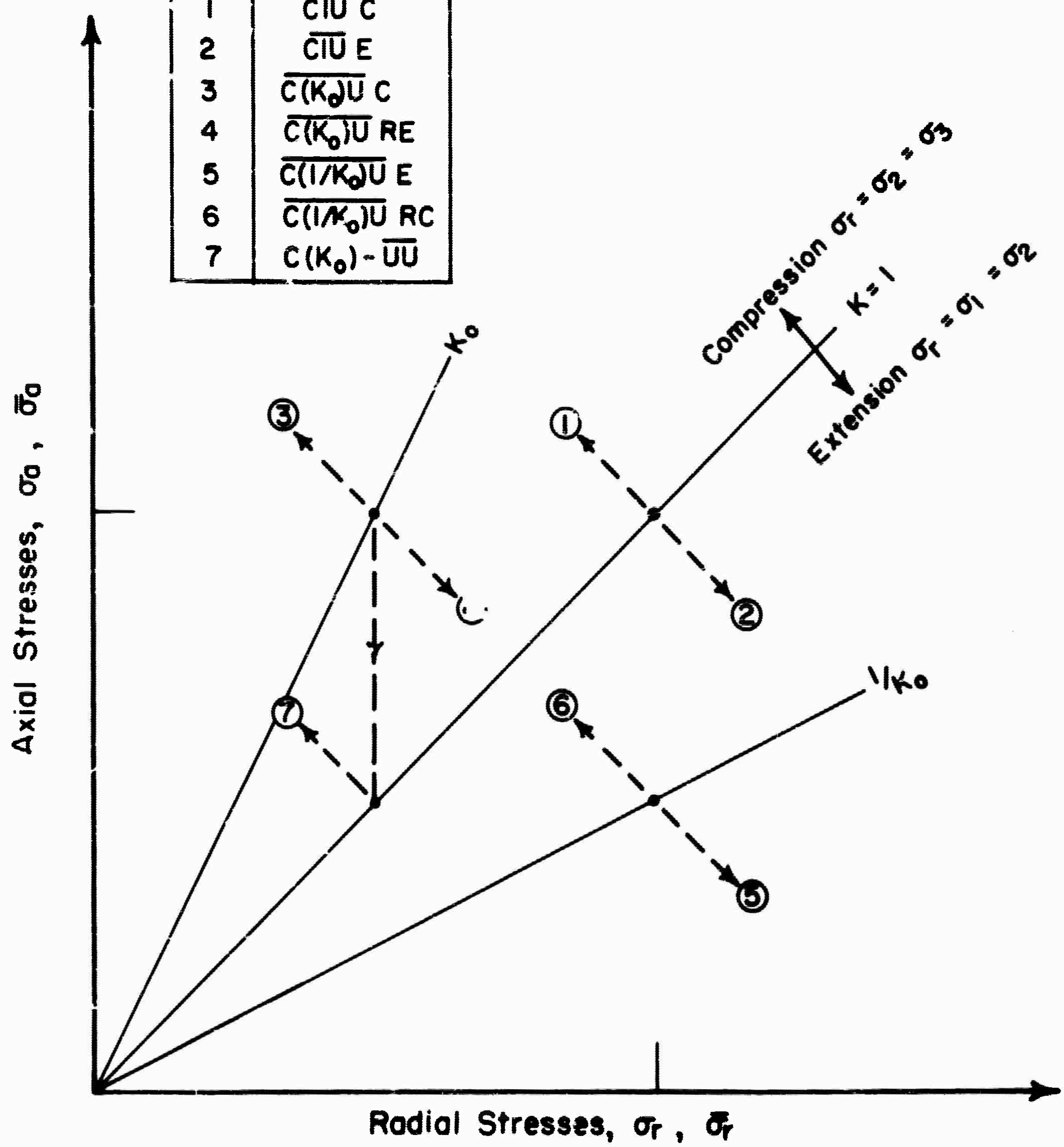


Fig.1.6 Stress Paths Employed for Experimental Program

$\overline{CU}$ Triaxial Tests on Normally Consolidated Boston Blue Clay

No.	Type of Test
1	$\overline{CIU}$ C
2	$\overline{CIU}$ E
3	$\overline{C(K_o)U}$ C
4	$\overline{C(K_o)U}$ RE
5	$\overline{C(1/K_o)U}$ E
6	$\overline{C(1/K_o)U}$ RC
7	$C(K_o)-\overline{UU}$



CHAPTER 2

REVIEW AND ANALYSIS OF PREVIOUS WORK ON THE INFLUENCE OF STRESS SYSTEM ON THE UNDRAINED STRENGTH BEHAVIOR OF SATURATED CLAYS

2.1 BASIC STRENGTH PRINCIPLES

Part I of this report presented a detailed explanation of the following three principles:

Principle I

For normally consolidated samples, or for overconsolidated samples with the same maximum past pressure $\bar{\sigma}_{cm}$, there is an unique relationship between strength and effective stress at failure (considering shear in compression and extension separately).

Principle II

For normally consolidated samples, or for overconsolidated samples with the same maximum past pressure, there is an unique relationship among water content, shear stress, and effective stress (considering shear in compression and extension separately).

Principle III

For both normally consolidated and overconsolidated samples, there is an unique relationship among strength, water content at failure and effective stress at failure as expressed by the Hvorslev parameters (considering shear in compression and extension separately).

These principles were illustrated by data on the hypothetical "Simple Clay," which showed that:

1. Maximum stress difference $(\sigma_1 - \sigma_3)$ and maximum obliquity of principal effective stresses $\bar{\sigma}_1/\bar{\sigma}_3$ were reached at the same strain in undrained shear tests;

2. Drained and undrained shear tests yielded the same effective stress envelope;

3. Effective stress paths for undrained shear after anisotropic consolidation ($\overline{CAU}$ tests) followed effective stress paths from undrained shear tests on isotropically consolidated samples ($\overline{CIU}$ tests);

4. Volume changes during drained tests could be deduced from effective stress paths obtained from $\overline{CIU}$ tests; conversely, stress paths for undrained tests could be deduced from the results of drained tests;

5. Compression and extension tests on normally consolidated samples yielded the same effective stress envelope, but differences in pore pressure ($\overline{CIU}$ tests) and volume changes (CID tests);

6. The principles only applied to tests wherein the shear stress was always increased, i.e., the application and removal of shear stresses were not considered.

As stated in Part I of this report, the actual strength behavior of clays often deviates from these principles. This chapter will look at the undrained strength behavior of actual clays as effected by stress system (anisotropic consolidation, perfect sampling, the value of the intermediate principal stress, and rotation of principal planes) based on information obtained from previous studies at M.I.T. and elsewhere.

2.2 EFFECT OF ANISOTROPIC CONSOLIDATION

2.2.1 <u>Theoretical Treatments</u>

Principles I and II state that stress paths from $\overline{CAU}$ tests should follow the stress paths from $\overline{CIU}$ tests and that water contents for anisotropic consolidation can also be obtained from results of $\overline{CIU}$ tests (see Figs. II-13 and II-14 of Part I). The basis for these principles was first proposed by Rendulic (1936) based on his tests on the Wiener Tegel clay. It was then hypothesized by Taylor (1948, footnote p. 387) and later determined experimentally by Henkel (1960a) for the remolded Weald clay.* It has also been used by Lowe and Karafiath (1960). The effect of anisotropic consolidation on the undrained strength behavior of the Simple Clay is shown below (for compression tests):

	$\overline{CIU}$ Tests	$\overline{CK_oU}$ Tests
$s_u/\bar{\sigma}_{1c}$	0.290	0.250
A_f	0.945	2.01
$\bar{\Phi}$	23.0^o	23.0^o

and $K_o = 1 - \sin\bar{\Phi} = 0.608$.

Skempton and Bishop (1954) assumed that A_f and $\bar{\Phi}$ were unchanged by anisotropic consolidation** and thus could calculate the ratio $s_u/\bar{\sigma}_{1c}$ for various values of K from the equation:***

$$\frac{s_u}{\bar{\sigma}_{1c}} = \frac{[K + A_f(1 - K)] \sin\bar{\Phi}}{1 + (2A_f - 1)\sin\bar{\Phi}} \tag{2.1}$$

The equation assumes $\bar{c} = 0$ and that the direction of σ_1 remains unchanged. If A_f and $\bar{\Phi}$ remain unchanged, then anisotropic

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* However, subsequent tests (Henkel and Sowa, 1963) on the Weald clay showed significant discrepancies.

** Hansen and Gibson (1948) also treated anisotropic consolidation, but employed the λ theory.

*** See p. 32 of Part I for a derivation of this equation for K = 1. Fig. II-16 of Part I plots $s_u/\bar{\sigma}_{1c}$ versus $\bar{\Phi}$ for K = 1 and $K_o = 1 - \sin\bar{\Phi}$ and for A_f = 0.5, 1.0 and 2.0.

consolidation increases $s_u/\bar{\sigma}_{1c}$ if A_f is greater than one, has no effect if $A_f = 1$, and decreases $s_u/\bar{\sigma}_{1c}$ if A_f is less than one.

Bjerrum and Lo (1961 and 1963) suggested the use of the following equation to correlate the results of $\overline{CIU}$ and $\overline{CAU}$ compression tests (for $\Delta\sigma_3 = 0$):

$$\frac{(\sigma_1 - \sigma_3)}{\bar{\sigma}_{1c}} = \left[\frac{\bar{\sigma}_1}{\bar{\sigma}_3} - 1\right]\left[1 - \left(\frac{\Delta u}{\bar{\sigma}_{1c}} + 1 - K\right)\right] \tag{2.2}$$

If $K = 1$, then:

$$\frac{(\sigma_1 - \sigma_3)}{\bar{\sigma}_{1c}} = \left[\frac{\bar{\sigma}_1}{\bar{\sigma}_3} - 1\right]\left[1 - \frac{\Delta u}{\bar{\sigma}_{1c}}\right] \tag{2.3}$$

The first term on the right-hand side of these equations is called the "strength term" and the second the "effective stress term."

Equation 2.2 implies that for a given value of $[\bar{\sigma}_1/\bar{\sigma}_3 - 1]$, the values of $(\sigma_1 - \sigma_3)/\bar{\sigma}_{1c}$ and $[1 - (\Delta u/\bar{\sigma}_{1c} + 1 - K)]$ will be independent of the preshear value of K. An analysis of the equation shows, however, that such will be the case only if the pore pressure parameter A is always equal to unity. This is illustrated in Fig. 2.1 for a hypothetical clay with $\bar{\Phi} = 30^{\circ}$ and $K_o = 1 - \sin\bar{\Phi} = 0.5$. The figures show that $\overline{CIU}$ and $\overline{CK_oU}$ tests yield the same curves only when $A = 1.0$.

2.2.2 Experimental Data

The effect of anisotropic consolidation on the undrained strength behavior of three remolded and three undisturbed normally consolidated (N.C.) clays is summarized in Table 2.1.* Plots of

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* Unless otherwise noted, all tests were performed at M.I.T.

consolidation pressure versus A_f and s_u and effective stress envelopes for these six clays are presented in Figs. 2.2 through 2.7. Effective stress paths are shown for some of the tests run at M.I.T. Figure 2.7 for the Weald clay also contains data from $\overline{\text{CAU}}$ tests on overconsolidated samples.

Figures 2.8 and 2.9 compare stress-strain curves for $\overline{\text{CIU}}$ and $\overline{\text{CAU}}$ tests on normally consolidated specimens of the remolded Boston blue clay and the undisturbed Kawasaki clays.

Other references containing information of the effects of anisotropic consolidation are:

Broms and Ratnam (1963) - $\overline{\text{CU}}$ tests with isotropic and anisotropic consolidation, employing a hollow cylinder shear device, on remolded kaolinite;

Henkel and Sowa (1963) - $\overline{\text{CIU}}$ and $\overline{\text{CK}_o\text{U}}$ triaxial tests on N.C. and O.C. remolded Weald clay;

Ladd, (1965) - review and analysis of $\overline{\text{CIU}}$ and $\overline{\text{CAU}}$ triaxial test data on the six clays in Table 2.1;

Landva (1962) - $\overline{\text{CIU}}$, $\overline{\text{CAU}}$, CID and CAD triaxial tests on N.C. undisturbed quick clay from Manglerud, Oslo, Norway;

Lo (1962) - $\overline{\text{CAU}}$ triaxial tests on specimens of undisturbed and remolded Mexico City cl

Lowe and Karafiath (1960) - $\overline{\text{CIU}}$ and $\overline{\text{CAU}}$ triaxial tests on compacted samples of core material for two dams;

Schmertmann and Hall (1961) - CFS tests on isotropically and anisotropically consolidated specimens of remolded kaolinite and Boston blue clay;

Simons (1960) - $\overline{\text{CIU}}$, $\overline{\text{CAU}}$, CID and CAD triaxial tests on O.C. undisturbed samples of the Brobekkveien, Oslo clay, but employing values of K corresponding to K_o for normally consolidated rather than overconsolidated specimens;

Simons (1963) - summary of effect of anisotropic consolidation on the effective stress envelopes of five undisturbed clays;

Whitman (1960) - quotes $\overline{\text{CIU}}$ and $\overline{\text{CAU}}$ triaxial test data on N.C. undisturbed samples of Boston blue clay from Taylor (1955).

2.2.3 Discussion

The test data on the six normally consolidated clays in Table 2.1 show the following effects of anisotropic consolidation:

At $(\sigma_1 - \sigma_3)_{max.}$

1. The change in $s_u/\bar{\sigma}_{1c}$ was generally small with a maximum increase of 10% and a maximum decrease of 15% (Table 2.1, Figs. 2.2 through 2.7);

2. The value of A_f decreased by a significant amount (0.2 to 0.5) except for the remolded Weald and Vicksburg Buckshot clays (Table 2.1, Figs. 2.2 through 2.7);

3. The slope of the effective stress envelope $\bar{\Phi}$ decreased by 0 to 4^{o} (Table 2.1, Figs. 2.2 through 2.7);

4. The strain at failure ϵ_f was considerably smaller, being generally less than 1% versus 2 to 15% for the $\overline{\text{CIU}}$ tests.

At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max.}$

5. The value of $q/\bar{\sigma}_{1c}$ was always decreased, often by a substantial amount (Table 2.1, Figs. 2.8 and 2.9), indicating that anisotropic consolidation produces a more sensitive structure;

6. The pore pressure parameter A generally increased by a substantial amount (Figs. 2.8 and 2.9), and in some cases even became negative [i.e., $(\sigma_1 - \sigma_3)$ became less than $(\sigma_1 - \sigma_3)$ at consolidation];

7. The value of $\bar{\Phi}$ was essentially unchanged (Table 2.1;

8. The value of $\Delta u/\bar{\sigma}_{1c} + (1 - K)$ was higher in the $\overline{CAU}$ tests, since $q/\bar{\sigma}_{1c}$ was decreased for a constant value of $\bar{\Phi}$.

In summary, for these six normally consolidated clays for which $\overline{CIU}$ tests yielded $s_u/\bar{\sigma}_{1c} = 0.29 - 0.45$, $A_f = 0.80 - 1.10$, and $\bar{\Phi} = 24 - 37^o$ [all at $(\sigma_1 - \sigma_3)_{max.}$], anisotropic consolidation decreased A_f and $\bar{\Phi}$ at $(\sigma_1 - \sigma_3)_{max.}$, and produced a more sensitive structure, but had little effect on $s_u/\bar{\sigma}_{1c}$ and on $\bar{\Phi}$ at maximum obliquity.

Figures 2.10 and 2.11 show that the terms in Eq. 2.2 are highly dependent on the value of K and hence the assumption that the relationship among shear stress, obliquity and effective stress is unique is not even approximately valid, except possibly at maximum obliquity.

The preceding data have shown that natural clays, as opposed to the Simple Clay, do not follow the strength principles regarding the influence of anisotropic consolidation on undrained strength behavior. This fact is illustrated by the stress paths plotted in Fig. 2.12 from $\overline{CIU}$ and $\overline{CAU}$ tests on the Kawasaki clay. The stress paths from the $\overline{CAU}$ tests obviously do not follow an extension of stress paths from $\overline{CIU}$ tests, thus negating Principle II. Principle I is generally valid at maximum obliquity but not at $(\sigma_1 - \sigma_3)_{max.}$ where $\overline{CAU}$ tests yield lower values of $\bar{\Phi}$. Moreover, the use of Principle II to calculate values of A_f and $s_u/\bar{\sigma}_{1c}$ for $\overline{CAU}$ tests from the results of $\overline{CIU}$ tests yields values of $s_u/\bar{\sigma}_{1c}$ which are generally much too low and values of A_f which are generally much too high (the Weald clay in Table 2.1 is an exception), as shown in Table 2.2. On the other hand, the use of Eq. 2.1 and values of A_f and $\bar{\Phi}$ from $\overline{CIU}$ tests do yield reasonable values of $s_u/\bar{\sigma}_{1c}$ due to the fact that the errors in the assumed values of A_f and $\bar{\Phi}$ are partially self-compensating (A_f too high causing an underestimate of strength and $\bar{\Phi}$ too high causing an overestimate of strength).

2.3 EFFECT OF PERFECT SAMPLING

2.3.1 Definitions and Equations

A sample of saturated clay is consolidated one-dimensionally and exists under K_o stresses so that:

Vertical consolidation pressure = $\bar{\sigma}_{vc}$

Horizontal consolidation pressure = $\bar{\sigma}_{hc} = K_o \bar{\sigma}_{vc}$.

As was shown in Fig. 1.2, K_o is approximately equal to 0.6 for normally consolidated clays and K_o becomes greater than unity (i.e., $\bar{\sigma}_{hc}$ becomes larger than $\bar{\sigma}_{vc}$) when the overconsolidation ratio exceeds approximately 3.5 ± 1. Perfect sampling denotes an undrained release of the K_o shear stress to attain an isotropic state of stress.

The isotropic effective stress after perfect sampling, $\bar{\sigma}_{ps}$, of a saturated clay which had vertical and horizontal consolidation pressures of $\bar{\sigma}_{vc}$ and $\bar{\sigma}_{hc} = K_o \bar{\sigma}_{vc}$ respectively can be derived as follows: Let $\Delta\sigma_v$ and $\Delta\sigma_h$ be the changes in vertical and horizontal total stresses* to achieve isotropic stress, so that:

$$(\Delta\sigma_v - \Delta\sigma_h) = -(\bar{\sigma}_{vc} - \bar{\sigma}_{hc}) = -\bar{\sigma}_{vc}(1 - K_o)$$

and let the resultant pore pressure change be Δu. Define the pore pressure parameter for unloading as A_u where:

$$A_u = \frac{\Delta u - \Delta\sigma_h}{\Delta\sigma_v - \Delta\sigma_h} \tag{2.4}$$

Since $(\Delta\sigma_v - \Delta\sigma_h) = -\bar{\sigma}_{vc}(1 - K_o)$, $\Delta u = \Delta\sigma_h - A_u \bar{\sigma}_{vc}(1 - K_o)$, the isotropic effective stress $\bar{\sigma}_{ps}$ will therefore be equal to:

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* Increases in σ_v and σ_h are positive values of $\Delta\sigma_v$ and $\Delta\sigma_h$ and decreases are negative values of $\Delta\sigma_v$ and $\Delta\sigma_h$.

$$\bar{\sigma}_{ps} = \bar{\sigma}_{hc} + \Delta\sigma_h - \Delta u$$

$$= K_o\bar{\sigma}_{vc} - A_u(\Delta\sigma_v - \Delta\sigma_h)$$

$$= \bar{\sigma}_{vc}K_o + A_u\bar{\sigma}_{vc}(1 - K_o)$$

$$\bar{\sigma}_{ps} = \bar{\sigma}_{vc}[K_o + A_u(1 - K_o)] \tag{2.5}$$

Equation 2.5 is valid for values of K_o both less than and greater than unity. Skempton (1961) and Seed, Noorany and Smith (1964) present equations similar to Eq. 2.5, but their form of A_u uses $\Delta\sigma_1$ and $\Delta\sigma_3$, which changes in direction when K_o becomes greater than unity.

The relationship between $\bar{\sigma}_{ps}$ and $\bar{\sigma}_{vc}$ is illustrated in Fig. 2.13 for normally consolidated (Point A) and highly overconsolidated (Point B) samples of a hypothetical clay for three different values of A_u. The straight line from the origin to Point A indicates a constant K_o of 0.65 for normally consolidated clay; the curved line from Point A to Point B shows an increasing value of K_o as the O.C.R. increases (K_o = 2.2 for O.C.R. = 10 at Point B). The figure shows that $\bar{\sigma}_{ps}/\bar{\sigma}_{vc}$ for normally consolidated clay will always be less than unity for A_u values less than one; the reverse is true for overconsolidated samples with $K_o \geq 1$, i.e., $\bar{\sigma}_{ps}/\bar{\sigma}_{vc}$ will be greater than unity for A_u values less than one.

Prior to running a laboratory triaxial shear test on so-called undisturbed samples, the clay must of course be removed from the ground, taken to the laboratory, trimmed and finally mounted in the test apparatus. Perfect sampling represents the best sample that can be tested because no disturbance has been given to the sample other than that involved with the release of the in situ shear stresses.

The perfect sampling process is also of interest in studying the effects of rotation of principal planes. Figure 1.5 showed that the clay beneath the center line of a circular excavation (effective stress path A'C') crossed over the K = 1 line (isotropic effective stresses) in the process of reaching failure. This type of undrained shear therefore represents a triaxial extension test on a "perfect" sample.

2.3.2 Experimental Data

Data on values of A_u and $\bar{\sigma}_{ps}/\bar{\sigma}_{vc}$ for several normally consolidated and overconsolidated clays are presented in Table 2.3. (Bishop and Henkel (1953), Seed, Noorany and Smith (1964) and Skempton and Sowa (1963) present additional data.) For normally consolidated clays, A_u generally equals $+0.15 \pm 0.15$. Since $K_o = 0.6 \pm 0.2$, the resulting value of the ratio of effective stress after perfect sampling to the vertical consolidation pressure is:

$$\bar{\sigma}_{ps}/\bar{\sigma}_{vc} = 0.66 \; (0.40 - 0.86) \; .$$

As clays become overconsolidated, the values of both A_u and K_o increase and hence the ratio $\bar{\sigma}_{ps}/\bar{\sigma}_{vc}$ increases. In fact, perfect sampling of heavily overconsolidated clays can yield values of $\bar{\sigma}_{ps}$ which exceed the vertical consolidation pressure $\bar{\sigma}_{vc}$.

Tables 2.4 and 2.5 present data on the effects of perfect sampling on the undrained strength behavior for triaxial compression for normally consolidated specimens of remolded Boston blue clay and of undisturbed Kawasaki clays. The $\overline{\text{CAU}}$ tests are undrained triaxial compression tests on anisotropically consolidated samples; the CA-$\overline{\text{UU}}$ tests are undrained triaxial compression tests on perfect samples. Figures 2.14 and 2.15 show stress paths and stress-strain curves for $\overline{\text{CAU}}$ and CA-$\overline{\text{UU}}$ tests on the Kawasaki clay.

The data on the Boston blue clay and the Kawasaki clay are summarized in Table 2.6. Similar data on undisturbed samples of the San Francisco Bay mud (from Seed, Noorany and Smith, 1964) and on remolded samples of the Weald clay (from Skempton and Sowa, 1963) are also summarized in this table.

2.3.3 Discussion

Perfect sampling had the following influence on the undrained strength behavior of the four normally consolidated clays shown in

Table 2.6, i.e., comparing CA-$\overline{UU}$ and $\overline{CAU}$ tests:

<u>At $(\sigma_1 - \sigma_3)_{max.}$</u>

1. The undrained strength s_u was decreased by 7% (range = 2 to 10%);
2. The pore pressure parameter A_f was decreased by 45% (range = 22 to 76%);
3. The strain at failure ϵ_f was increased by 2 to 3 fold (range = 1.15 to 6.1 times larger),
4. The slope of the effective stress envelope $\overline{\Phi}$ was increased by 1.1^o (range = 0.6 to 1.7^o).

<u>At $(\overline{\sigma}_1/\overline{\sigma}_3)_{max.}$</u> (BBC and Kawasaki only)

5. The shear strength was slightly lower (ave. decrease = 4 ± 2%);
6. The effective stress envelope was unchanged.

In summary, perfect sampling causes a slight decrease in undrained shear strength, has no influence on the effective stress envelope at maximum obliquity and only a slight influence at $(\sigma_1 - \sigma_3)_{max.}$, but causes a large reduction in the pore pressure parameter A_f.

2.4 EFFECT OF INTERMEDIATE PRINCIPAL STRESS

2.4.1 Theory

The undrained shear strength s_u of an isotropically consolidated saturated sample is related to consolidation pressure $\overline{\sigma}_c$, pore pressure parameter at failure A_f, and friction angle $\overline{\Phi}$ (for $\overline{c} = 0$) by (see Eq. 2.1, set K = 1):

$$\frac{s_u}{\overline{\sigma}_c} = \frac{\sin \overline{\Phi}}{1 + (2A_f - 1)\sin \overline{\Phi}} \tag{2.6}$$

The value of the intermediate principal stress σ_2 during undrained shear can therefore influence the value of s_u in two ways: by changing the effective stress envelope and by changing the excess pore pressure and hence A_f.

The Mohr-Coulomb failure criteria assumes that σ_2 has no effect on the failure envelope, whereas other failure criteria, such as the extended Tresca and the extended von Mises theories, predict that $\bar{\phi}$ will be increased by increased values of σ_2 (see Hvorslev, 1960, for an extensive discussion of different failure criteria and for a review of experimental data).

Henkel (1960b) has suggested a pore pressure equation in terms of the octahedral stresses, rather than simply in terms of changes in the major and minor principal stresses, to account for the influence of σ_2 on excess pore pressures:

$$\Delta u = \frac{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3}{3} + a[(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_1 - \Delta\sigma_3)^2]^{1/2} \tag{2.7}$$

If the revised parameter "a" is independent of $\Delta\sigma_2$, the value of Δu for a given value of $(\Delta\sigma_1 - \Delta\sigma_3)$ will increase with increasing values of $\Delta\sigma_2$. As stated on p. 54 of Part I, the pore pressure parameter A for extension tests will be equal to that for compression tests plus 1/3 at the same value of $(\Delta\sigma_1 - \Delta\sigma_3)$ if "a" is constant.

Juarez-Badillo (1963) has employed octahedral normal and shearing stresses in his analysis of excess pore pressures in terms of changes in stress difference for consolidated-undrained compression and extension tests on normally consolidated and overconsolidated clays.

2.4.2 Experimental Data

The results of consolidated-undrained shear tests with pore pressure measurements on saturated clays employing varying values of the intermediate principal stress are summarized below. Unless otherwise stated, all tests were isotropically consolidated.

Taylor (1955) summarized the results of $\overline{CIU}$ triaxial compression and extension tests on normally consolidated and overconsolidated samples of undisturbed Boston blue clay (P.I. = 17-29%). A valid numerical comparison of the data is not possible because the compression and extension tests were run on different batches of the clay, although trends were established. For normally consolidated samples, the extension tests yielded values of s_u some 10 to 20% lower, but the same friction angle. Similar decreases in s_u occurred with overconsolidated samples, but the effective stress envelope appeared to be somewhat higher. Taylor concluded that the decrease in s_u for the extension tests was caused by an increase in excess pore pressures.

Hirschfeld (1958) ran $\overline{CIU}$ triaxial compression and extension tests on three normally consolidated undisturbed clays (an inorganic clay with P.I. = 25% and two organic silty clays with P.I. = 24 and 28%). Values of s_u for extension were 20 to 25% lower than those for compression; there was too much scatter in the data to detect any significant change in the friction angle

Parry (1960) reports the results of an extensive series of $\overline{CIU}$ triaxial compression and extension tests on normally consolidated and overconsolidated samples of remolded Weald clay (w_L = 43%, P.I. = 25%, mixing w = 34%). These data are summarized in Fig. 2.16. Extension tests produced undrained strengths about 15% below those failed in compression and increased A_f by 0.23 ± .05. The value of $\overline{\Phi}$ was unchanged for normally consolidated samples, whereas failure in extension appeared to cause a slight increase in the effective stress envelope of heavily overconsolidated specimens. Axial strains at failure for the extension tests were only about one-half of those which occurred in the compression tests.

Wu, Loh and Malvern (1963) performed $\overline{CIU}$ triaxial compression and extension tests and $\overline{CIU}$ hollow cylinder tests with varying values of σ_2 on normally consolidated samples of remolded Sault Ste Marie clay (a glacial lake clay with $w_L = 52 - 56\%$, P.I. = 24-29% and mixing w = 40%). Strength parameters for extension and compression are shown in Fig. 2.17. Plots of s_u versus $\bar{\sigma}_c$ and q_f versus $\bar{p}_f$ for all test data are presented in Figs. 2.18 and 2.19 respectively. These data presumedly represent conditions at maximum stress differences, although they may also closely approximate maximum obliquity because the soil is insensitive. Table 2.7 gives average values of the various parameters from tests having a consolidation pressure greater than 2.6 kg./cm^2. Although there are considerable scatter in some of the data, the following trends appear evident:

1. There is little effect of σ_2 on $s_u/\bar{\sigma}_c$ until $\bar{\sigma}_{2f}$ becomes appreciably greater than $\bar{p}_f = (\bar{\sigma}_1 + \bar{\sigma}_3)_f/2$. Comparing the extremes, the strength in triaxial extension was 30% less than the strength in triaxial compression;

2. The friction angle remains unchanged unless the test results of I_1 and I_3 are considered significant;

3. Consequently, the decrease in s_u at very large values of $\bar{\sigma}_{2f}$ is caused solely by increased excess pore pressures.

However, the results from the hollow cylinder tests may well have been influenced by experimental problems (as stated by the authors in their closure to discussion - ASCE, JSMFD, Vol. 90, SM2, p. 165, March, 1964). For example, test series C1 (triaxial compression) and C1a (hollow cylinder compression) should have yielded identical results whereas measured values of $s_u/\bar{\sigma}_c$ and $\Delta u/\bar{\sigma}_c$ often differed more than did the results from the hollow cylinder tests wherein σ_2 was varied appreciably.

Broms and Casbarian (1964) present the results of $\overline{CIU}$ hollow cylinder tests, with widely varying values of σ_2, on normally consolidated remolded kaolinite (w_L = 57%, P.I. = 25%, Activity = 0.64, mixing w = 48.5%). These data are summarized in Fig. 2.20 wherein $s_u/\bar{\sigma}_c$, A_f and $\bar{\Phi}$ are plotted against $(\sigma_2 - \sigma_3)_f/(\sigma_1 - \sigma_3)_f$ for three values of consolidation pressure. The ratio $(\sigma_2 - \sigma_3)_f/(\sigma_1 - \sigma_3)_f$ expresses σ_{2f} in terms of its location between σ_{1f} and σ_{3f}.* The ratio is zero for triaxial compression ($\sigma_{2f} = \sigma_{3f}$) and is unity for triaxial extension ($\sigma_{2f} = \sigma_{1f}$). The data show that as σ_{2f} increases from σ_{3f} (triaxial compression) to σ_{1f} (triaxial extension): 1) the value of A_f increases; 2) $\bar{\Phi}$ increases until σ_{2f} is about halfway between σ_{3f} and σ_{1f} and then remains constant; and 3) $s_u/\bar{\sigma}_c$ remains approximately constant until σ_{2f} is halfway between σ_{3f} and σ_{1f} and then undergoes a fairly large decrease which reaches 25 ± 5% when $\sigma_{2f} = \sigma_{1f}$ (triaxial extension).

Bishop (1957 and 1961) quotes comparisons of effective stress envelopes from $\overline{CU}$ triaxial compression and plane strain tests on K_o consolidated samples of a compacted moraine (3% minus 2μ). The plane strain sample was 4 in. high by 2 in. deep by 16 in. long (see Cornforth, 1964, for a detailed description). Values of $\bar{\Phi}$ were 2^o and 4^o higher in plane strain than in triaxial compression at conditions of maximum stress difference and maximum obliquity respectively. The cohesion intercept $\bar{c}$ was increased by 1.7 and 1.0 psi respectively. For plane strain, the value of $\bar{\sigma}_{1f}$ was approximately equal to $0.3(\bar{\sigma}_1 + \bar{\sigma}_3)_f$.

Wade (1963) performed an extensive series of $\overline{CU}$ plane strain tests (same equipment as above) on K_o consolidated samples

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* The above total stresses can, of course, be replaced by effective stresses.

of remolded normally consolidated Weald clay and compared his results with those obtained from $\overline{CK_oU}$ triaxial compression tests on the same soil (Skempton and Sowa, 1963). Relative to the triaxial compression tests, the plane strain tests produced a slight increase in $s_u/\bar{\sigma}_{1c}$ (0.28 vs. 0.26); a slight increase in $\bar{\Phi}$ at both maximum stress difference and maximum obliquity (by 1.2^o); a large decrease in A_f (1.64 ± 0.10 vs. 2.15), although the value of $\Delta u/\bar{\sigma}_{1c} + (1 - K_o)$ was only slightly decreased (0.65 ± 0.10 vs. 0.66); and a large decrease in the strain at failure (2 - 3% vs. 5 - 6%). The intermediate principal stress at failure was less than the average of the other two principal stresses ($\bar{\sigma}_{2f} = 0.41\ \bar{p}_f$).

2.4.3 Discussion

Table 2.8 summarizes the effects of the intermediate principal stress on the undrained strength behavior of saturated clays. Extension tests, relative to compression tests, employing both triaxial and hollow cylinder shear devices, show very consistent trends in that:

1. $s_u/\bar{\sigma}_c$ always decreases by 20 ± 10%;
2. A_f always increases by 0.4 ± 0.2;
3. $\bar{\Phi}$ of normally consolidated clays remains essentially unchanged (except for the remolded kaolinite wherein $\bar{\Phi}$ increased by 7^o);
4. The effective stress envelope of heavily overconsolidated clays showed a slight increase.

However, when the value of σ_2 is midway between that for compression and extension, and/or equal to that for plane strain, the experimental data show that the value of undrained strength is little different than that from triaxial compression tests, although the friction angle generally increased. Changes in A_f were erratic and varied from large increases (remolded kaolinite) to large decreases (remolded Weald clay).

There are two major drawbacks in extrapolating the data for intermediate values of σ_2 to field practice. First, the data obtained from the hollow cylinder tests are difficult to interpret because: 1) nonuniform stress distributions and end restraint have effects on the data; and 2) one has to assume the validity of the theories of elasticity and plasticity and a failure criterion in order to compute two of the three principal stresses. Second, the plane strain data, although free from problems of interpretation, were obtained on anisotropically consolidated samples of a clay whose undrained strength behavior is apparently considerably different than that of natural normally consolidated clays (see Section 2.2 on effect of anisotropic consolidation).

2.5 EFFECT OF ROTATION OF PRINCIPAL PLANES

2.5.1 General

Section 1.3 pointed out examples in the field where rotation of principal plan occurred during shear. The rotation of principal planes along the failure arc resulting from undrained failure of a strip footing resting on a normally consolidated clay is depicted in Fig. 2.21. In element A under the footing, the direction of the major principal stress at failure, $\bar{\sigma}_{1f}$, coincides with the direction of the major principal stress at consolidation, $\bar{\sigma}_{1c}$, i.e., there is no rotation. In element B, $\bar{\sigma}_{1f}$ rotates $45 + \bar{\Phi}/2$ degrees* to the left for a horizontal failure surface, and in element C, the major principal stress rotates 90° to the left. Another way of describing the amount of rotation is to look at the direction of the failure plane relative to the direction of major principal stress at consolidation.

Figure 2.22 shows the direction of the principal planes before and after shear of a normally consolidated clay via an in situ vane shear test. The failure plane is vertical and the directions

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* Assuming validity of the Mohr-Coulomb failure criteria.

of the major and intermediate principal stresses have rotated by 90°.

The change in direction of principal planes during a direct shear test is shown in Fig. 2.23. The failure plane is presumed to be horizontal so that the major principal stress rotates $45 + \overline{\phi}/2$ degrees, just as for element B in Fig. 2.21.

If the clay had isotropic properties, the direction of the failure plane would have no influence on strength behavior. However, the in situ preshear stress system is seldom isotropic; therefore, various planes through the soil have different consolidation stresses and probably different orientations of the clay particles, and consequently one might expect to obtain different strengths along different failure planes. The question is: how significant is the direction of the failure plane* on the undrained strength behavior?

2.5.2 Theoretical Treatments

Hansen and Gibson (1948) employed Skempton's λ theory (Skempton, 1948) and the Hvorslev parameters to compute the variation in undrained shear strength, s_u, of saturated clay with inclination of the in situ failure plane, including the in situ vane tests with its vertical failure surface. They also computed theoretical values for laboratory UU and CU tests with triaxial compression and direct shear (horizontal failure plane) equipment based on two extremes of sampling (the case of perfect sampling and the case wherein the clay underwent a passive failure prior to sampling). The results of their analysis for a hypothetic sensitive silty clay with $K_o = 0.50$ are tabulated below:

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* There can be a subtle difference between "direction of failure plane" and "rotation of principal planes," but this difference will be ignored in view of the general lack of information on this topic.

Test Condition		Values of $s_u/\bar{\sigma}_{vo}$
In Situ:	Active earth pressure	0.331
	Passive earth pressure	0.193
	Horizontal failure plane	0.213
	Field vane	0.191
Lab.	Unconfined	
	Perfect sample	0.250
	"Failed" sample	0.170
	Consolidated-Undrained	
	Triaxial compression	0.348
	Direct shear	0.213

These theoretical estimates show that the direction of the failure plane is a very important consideration. Data will be shown to support the large differences indicated above. The predictions of Hansen and Gibson are indeed surprisingly accurate in view of the greatly oversimplified picture of soil properties that was assumed. For example, the compressibility of soil in all three directions was assumed to be linear and equal, the Hvorslev parameters were assumed to be unique, and $\Delta\bar{\sigma}_2$ was assumed equal to zero for plane strain.

Schmertmann (1964) suggests that the undrained shear strength along various planes through an anisotropically consolidated clay is proportional to the preshear consolidation stress on the plane of failure. Consequently, he predicts that s_u along a vertical failure plane (such as from a field vane) will equal K_o times s_u along a horizontal plane (for vertical one-dimensional consolidation). For the hypothetical clay treated by Hansen and Gibson (1948), Schmertmann would therefore predict s_u (vertical plane) = 0.50 s_u (horizontal plane), whereas Hansen and Gibson predict s_u (vertical plane) = 0.90 s_u (horizontal plane).

Tenny (1960) and Hansen (1963) have treated in situ vane strengths in horizontal K_o consolidated clays as equivalent to triaxial compression tests on samples isotropically consolidated to the in situ horizontal consolidation stress (i.e., $\bar{\sigma}_c = \bar{\sigma}_{ho} = K_o\bar{\sigma}_{vo}$).

2.5.3 Experimental Data

The types of tests desired for analysis are undrained shear tests on anisotropically consolidated clay with different directions of the failure plane at a constant value of σ_2.* The following data are from tests which fail to meet this requirement, but nevertheless show important trends which shed light on the problem.

Broms and Casbarian (1964) ran $\overline{CU}$ hollow cylinder tests on isotropically consolidated samples of remolded kaolinite. The axial (σ_z) and tangential (σ_θ) stresses were varied such that the radial stress (σ_r) equalling σ_2 was kept equal to the consolidation pressure. A torque was then applied to the top of the sample to produce a change in the direction of the principal planes and to cause an undrained failure. Their test data are plotted in Fig. 2.24. At $\alpha = 0^\circ$, σ_1 equalled σ_z (failure caused by increasing σ_z and decreasing σ_θ); at $\alpha = 90^\circ$, σ_1 equalled σ_θ (failure caused by increasing σ_θ and decreasing σ_z); at intermediate angles, a torque was applied to the sample. The data show essentially equal strength parameters at $\alpha = 0^\circ$ and $\alpha = 90^\circ$, as would be expected for an isotropically consolidated sample. At intermediate values of α, the undrained strength decreased because of increased excess pore pressures and a lower effective stress envelope. The maximum strength reduction (about 30%) occurred at $\alpha = 45^\circ$. Broms and Casbarian explain this strength minimum in terms of a rotation of σ_1 through an angle of 45°, so that the failure surface almost coincided with the orientation of the clay particles during the initial phase of the test. The writers disagree with this reasoning because a test with $\alpha = 45^\circ$ on an isotropically consolidated sample must be a simple shear test wherein the direction of σ_1 remains unchanged during shear. Nevertheless, the test data show a very significant influence of the direction of the failure plane. Perhaps some of the effect is

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* Or for σ_2 corresponding to plane strain.

caused by the assumptions which must be made in order to compute the principal stresses, or is due to problems in equipment calibration. Or perhaps the clay structure was not truely isotropic prior to shear.

The results of consolidated-undrained triaxial compression and extension tests on anisotropically consolidated samples of normally consolidated undisturbed Kawasaki Clay II are presented in Figs. 2.25 and 2.26 (Lambe, 1962b). Two samples were consolidated to essentially identical pressures. In the $\overline{\text{CAUC}}$ test, the axial stress was increased; in the $\overline{\text{CAURE}}$ test, the radial stress was increased. The tests correspond to tests 3 and 4 respectively, in Fig. 1.6, and to undrained shear under a circular footing and beneath a circular excavation (Fig. 1.4) respectively. The test results show:*

At $(\sigma_1 - \sigma_3)_{max.}$	$\overline{\text{CAUC}}$ Test (Circular Footing)	$\overline{\text{CAURE}}$ Test (Circular Excavation)
s_u (kg/cm^2)	1.33	0.68
$A_f = \frac{\Delta u}{\Delta \sigma}$	0.53	0.96
$\bar{\Phi}$, degrees	36.5	47?
Axial strain, %	1.2 (compression)	9.8 (extension)
ϵ in direction of $\Delta\sigma$, %	1.2	~5

The above data show that the stress system applied during shear had a very important effect on undrained shear strength (by a factor of two), excess pore pressures, friction angle and stress-strain behavior. It is not possible, however, to separate out the basic cause of the effect because there are two variables: 1) change in direction of the principal stress (the principal stress

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* Although the water contents of the two samples were very different, extensive shear data on this clay have shown that the strength parameters are essentially independent of water content variations.

rotated 90° in the $\overline{\text{CAURE}}$ test); and 2) different stress systems at failure (triaxial compression in the $\overline{\text{CAUC}}$ test versus triaxial extension in the $\overline{\text{CAURE}}$ test, i.e., the relative magnitude of σ_2 was different for the two tests).

The fact that rotation of principal planes per se will not always produce a large reduction in strength is illustrated by the triaxial test data in Figs. 2.27 and 2.28 (Whitman, Ladd and da Cruz, 1960). One sample was consolidated with the axial stress greater than the radial stress and then failed in undrained compression, i.e., a regular $\overline{\text{CAUC}}$ test. The second sample was consolidated with the radial stress greater than the axial stress and then failed undrained in compression by increasing the axial stress (denoted by $\overline{\text{CAURC}}$ and corresponds to test No. 6 in Fig. 1.6). The undrained strengths of the two samples are almost the same even though the pore pressure and stress-strain characteristics are markedly different. In this test series, as contrasted to the previous one, the stress system at failure was the same in both samples but the stress system at consolidation was different.

The Norwegian Geotechnical Institute (Landva, 1962) performed undrained triaxial compression and simple shear tests on anisotropically, normally consolidated undisturbed samples of a silty quick clay from Manglerud, Oslo, Norway. Pertinent data on the Manglerud clay are:

Depth of samples = 6 - 9.5 m.

w_L = 25 - 27%, P.I. = 5 - 8%, Activity = 0.11 - 0.17

L.I. = 2 - 3.5, Sensitivity $\geq$ 100

$\tau_{max.}/\bar{\sigma}_{vo}$ = 0.16 ± 0.05 from field vane

$s_u/\bar{\sigma}_{vo}$ = 0.23 from average of 5 unconfined tests

K_o = 0.50 ± 0.03 from triaxial tests.

The triaxial tests were regular strained controlled $\overline{\text{CK}_o\text{UC}}$ tests except that special efforts were made to minimize disturbance and the rate of strain was very low (only 0.1% axial strain per hour). The simple shear tests (called Consolidated Constant Volume Direct Shear Tests by N.G.I.) were run on cylindrical samples (dia. = 10 cm, height = 1 cm) in which the horizontal surface was prevented from tilting. Lateral deformation was restrained via a reinforced (steel wires) rubber membrane. During shear, the normal load was varied in order to maintain a constant volume. Hence the tests were undrained. The samples were consolidated in the apparatus prior to shear and presumedly had a K_o stress system.

The data from the triaxial and simple shear tests are compared in Figs. 2.29, 2.30 and 2.31. Note that measured values of stress have in some instances been adjusted in order to compare like parameters. For example, in Fig. 2.29, values of shear stress τ from the simple shear tests have been divided by cos $\bar{\Phi}$ in order to obtain $(\sigma_1 - \sigma_3)/2 = q$. Furthermore, excess pore pressures are compared in terms of $\Delta u' = \bar{\sigma}_{1c} - \bar{\sigma}_{ff}$, which is directly measured in the simple shear tests ($\Delta u'$ = change in $\bar{\sigma}$ on horizontal plane), but must be computed for the triaxial compression tests (see Fig. 2.31). The strain in the simple shear test is equal to the horizontal movement divided by sample thickness.

At maximum stress difference, the simple shear tests, relative to the triaxial compression tests (for $\bar{\sigma}_{1c}/\bar{\sigma}_{vo}$ values greater than 1.2):

1. Had a 25% lower undrained shear strength expressed as $q_{max.} = \tau/\cos \bar{\Phi}$
2. Had a friction angle some 5 - 6^o lower, which was the principal cause of the lower undrained strength;
3. Had a much higher strain (10% versus only 0.3% for the triaxial tests).

At maximum obliquity, the simple shear tests again yielded much lower strengths and friction angles.

The stress system in the two tests differ in two respects: 1) the triaxial tests are failed in triaxial compression (i.e., $\sigma_2 = \sigma_3$), whereas the simple shear tests fail approximately in plane strain $[(\sigma_2 \approx (\sigma_1 + \sigma_3)/2]$; and 2) the direction of the major and minor principal planes remain constant in the triaxial tests whereas they rotate through an angle of $45 + \bar{\phi}/2 \approx 55 - 60^{\circ}$ in the simple shear test (see Fig. 2.23). The data on the effect of σ_2 on strength behavior (see Table 2.8 for the summary) indicate that plane strain has relatively little effect on s_u, but may increase excess pore pressures and the friction angle somewhat. Although these data are for insensitive clays, there is no reason to believe that the trends would be completely reversed for the quick Manglerud clay. Consequently, the vastly different strength behavior of this clay in triaxial compression and in simple shear must be caused primarily by the rotation of the principal planes.

Landva (1962) also reports the results of miniature lab vane tests run inside triaxial samples for both isotropic and anisotropic consolidation. These data are summarized below. They should, however, be treated as preliminary results.

Manglerud Clay ($\bar{\sigma}_{1c}/\bar{\sigma}_{vo} \geq 2$)

Type of Test	$\tau/\bar{\sigma}_{1c}$	$s_u/\bar{\sigma}_{1c}$	$s_u/\bar{\sigma}_{fc}$*	Comments
Triaxial $\overline{\text{CIUC}}$	-	0.29	0.29	
Triaxial $\overline{\text{CAUC}}$	-	0.28	0.435	K = 0.50, $\bar{\phi} = 24.7^{\circ}$
Lab Vane, CIV	0.38	0.405	0.405	Assume $\bar{\phi} = 20^{\circ}$
Lab Vane, CAV**	0.19	0.20	0.355	K = 0.50, Assume $\bar{\phi} = 20^{\circ}$

* $\bar{\sigma}_{fc}$ = effective normal stress at consolidation on plane which ends up as the failure plane.

** $\bar{\sigma}_{fc}/\bar{\sigma}_{1c} = [K(H/D) + a/2]/[(H/D) + a/2]$ where H/D = height/diameter and "a" is a parameter expressing distribution of shear stress on horizontal ends of the vane (Schmertmann, 1964)

Skabo Clay ($\bar{\sigma}_{1c}/\bar{\sigma}_{vo} \geq 2$) (see Table 2.1 for index properties)

Type of Test	$\tau/\bar{\sigma}_{1c}$	$s_u/\bar{\sigma}_{1c}$	$s_u/\bar{\sigma}_{fc}$	Comments
Triaxial $\overline{\text{CIUC}}$	-	0.32	0.32	
Triaxial $\overline{\text{CAUC}}$	-	0.32	0.505	$\bar{\Phi} = 26.5^{\circ}$, K = 0.49
Lab Vane, CIV	0.53	0.565	0.565	Assume $\bar{\Phi} = 20^{\circ}$
Lab Vane, CAV	0.32	0.34	0.61	K = 0.49, Assume $\bar{\Phi} = 20^{\circ}$

For isotropic consolidation, the data show undrained strengths from the vane which are 60 ± 20% higher than triaxial compression strengths. This is certainly surprising and casts considerable doubt on the accuracy of the data. On the other hand, the vane data on the isotropically and anisotropically consolidated samples show remarkable agreement in terms of strength as a function of consolidation stress on the failure plane, i.e., values of $s_u/\bar{\sigma}_{fc}$. Moreover, these values are in reasonable agreement with values of $s_u/\bar{\sigma}_{fc}$ obtained from the $\overline{\text{CAUC}}$ tests.

In situ vane data by the N.G.I. from three Norwegian clays employing vanes of various height to diameter ratios have indicated strengths on the horizontal plane which are 50 - 60% higher than those on the vertical plane (from G. Aas of the N.G.I. during a visit to M.I.T. in September 1963). Two of the clays were very sensitive to quick, the third was moderately sensitive.

2.5.4 Discussion

The preceding data do not show directly the influence of rotation of principal planes on the strength behavior of anisotropically consolidated clays. However, there can be no doubt that the undrained strength, excess pore pressures, friction angle, and stress-strain characteristics of an anisotropically consolidated sensitive clay are greatly affected by the type of stress system applied during shear. Variations in undrained strengths of 25 - 50%, changes in friction angle of several degrees, and very large differences in the strain at failure could easily occur along a single failure arc of a strip footing or between clay elements beneath a circular footing versus a circular excavation. The clay most susceptable to these effects would probably be a normally consolidated sensitive to quick clay having a high degree of anisotropy. An overconsolidated clay with $K = 1$ would probably be little effected.

For a given clay, the most important variables are thought to be the preshear value of K, the direction of σ_1 at failure relative to its direction after consolidation, and the relative value of σ_2 during shear.

The belief that undrained strength can be uniquely related to the consolidation stress on the failure plane is certainly an oversimplification. For example, the preceding data have shown:

Normally Consolidated Kawasaki Clay ($K \approx K_o \approx 0.50$)

Failure In	$s_u/\bar{\sigma}_{1c}$	$s_u/\bar{\sigma}_{fc}$
Triaxial compression	0.445	0.745
Triaxial extension	0.225	0.24

Normally Consolidated Manglerud Clay ($K \approx K_o \approx 0.50$)

Failure In	$s_u/\bar{\sigma}_{1c}$	$s_u/\bar{\sigma}_{fc}$
Triaxial compression	0.28	0.435
Simple shear	0.21	0.21

Finally, the interpretation of field vane data in normally consolidated clays is questionable at best. Brinch Hansen (1963) and Kenney (1960) have treated vane strengths as equivalent to $\overline{\text{CIU}}$ compression tests starting from $\bar{\sigma}_c$ equal to the in situ horizontal stress. The preceding data do not support this contention.

Table 2.1 Effect of Anisotropic Consolidation on Consolidated-Undrained Triaxial Compression Tests on Normally Consolidated Clays (All stresses in kg/cm^2)

1	2	3	4	5	6	7	8	9	10	11	12	13
Soil	Classification Data				Consolidation		At $(\sigma_1 - \sigma_3)_{max.}$				At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max.}$	
	W_L, %	I_P, %	I_L, %	Activity	$\bar{\sigma}_{1c}$	$\bar{\sigma}_{3c}/\bar{\sigma}_{1c}$	$S_u/\bar{\sigma}_{1c}$	A_f	$\bar{\phi}°$	$\mathcal{E}_f$, %	$q/\bar{\sigma}_{1c}$	$\bar{\phi}°$
Remolded Boston Blue Clay (Fresh water) (S_t = 5-10)	33	15	375	0.5	4-6	1.00	0.30	1.10	27.5	2.5	0.29	32.5
					3-6	0.54	0.33	0.60	26.5	0.4	0.27	33
Remolded Weald Clay *	46	24	48	0.63	2-7.5	1.00	0.32	0.92	26	15	—	—
					2-7	0.61	0.27	1.80	26	7	—	—
Remolded Vicksburg Buckshot Clay	63	39	230	0.75	3-6	1.00	0.28	1.05	24	6	0.27	25
					3-6	0.54	0.28	1.05	23.5	0.7	0.25	25
Undisturbed Kawasaki Clays I and II (S_t = 10)	80 (50-100)	38 (20-50)	80 (60-100)	~1 (0.7-1.7)	1.5-6	1.00	0.45	0.80	37	4.5	0.44	38.5
					2-5.5	0.52	0.42	0.50	33	0.9	0.40	39
Undisturbed Brobekkvelen Oslo Clay ** (S_t = 5)	39	18	72	0.5	1.5-4	1.00	0.35	0.95	30.5	3.5	0.35	31
					2-12	0.47	0.32	0.75	27	0.6	0.28	31.5
Undisturbed Skabo Clay *** (S_t = 2-6)	52	29	65	0.6	2-6	1.00	0.32	1.05	30	5.3	—	—
					2-5	0.47	0.32	0.75	26.5	0.6	—	—

* From Skempton and Sowa (1963) ** From Simons (1960) *** From Landva (1962)

Notes:

Col. 1: S_t = sensitivity

Col. 4: I_L values at remolding and in situ for remolded and undisturbed samples respectively

Col. 5: Activity = I_p/(% - 2 microns)

Col 6: Range of values of $\bar{\sigma}_{1c}$ prior to shear

Col. 7-13: Ave values from tests within the pressure range in Col. 6

Col. 10, 13: Values of $\bar{\phi}$ assuming zero cohesion intercept

Col. 11 : $\mathcal{E}_f$ = axial strain at failure

Col. 12 : $q = (\sigma_1 - \sigma_3)/2$

TABLE 2.2

MEASURED VERSUS PREDICTED STRENGTH
PARAMETERS FOR $\overline{\text{CAU}}$ TESTS

Clay	Measured $s_u/\bar{\sigma}_{1c}$	Measured A_f	From Principle II* $s_u/\bar{\sigma}_{1c}$	From Principle II* A_f	From Eq. 2.1** $s_u/\bar{\sigma}_{1c}$
Boston Blue	0.3[illegible]	0.60	0.265	3.6	0.31
Vicksburg Buckshot	0.28	1.05	0.[illegible]5	[illegible].5	0.29
Kawasaki	0.42	0.50	0.35	1.2	0.40

* Assumes that $\overline{\text{CAU}}$ tests follow the effective stress paths from $\overline{\text{CIU}}$ tests.

** Assumes that anisotropic consolidation yields the same values of $\bar{\Phi}$ and A_f as obtained from $\overline{\text{CIU}}$ tests.

TABLE 2.3

STRESS RATIOS FOR PERFECT SAMPLING

$$\bar{\sigma}_{ps} = \bar{\sigma}_{vc} [K_o + A_u (1 - K_o)] \qquad A_u = \frac{\Delta u - \Delta \sigma_h}{\Delta \sigma_v - \Delta \sigma_h}$$

Clay	Atterberg Limits	O.C.R.*	No. of Tests	K_o	A_u	$\bar{\sigma}_{ps}/\bar{\sigma}_{vc}$	Reference
Undisturbed Kawasaki Clays	w_L = 48 - 106% PI = 16 - 46%	N.C.	3	~0.47	+0.15 (.07 - .28)	0.545 (.50 - .61)	Ladd and Lambe, 1963 and Table 2.5
Remolded Boston Blue Clay	w_L = 33% PI = 15%	N.C.	4	0.54 ±.01	+0.18 ±.06	0.62 ±.05	Table 2.4
Remolded Weald Clay	w_L = 46% PI = 24%	N.C.	4	0.60	-0.033 ±.02	0.59 ±.02	Skempton and Sowa, 1963
		2.2	1	0.80	+0.32	0.87	Henkel and Sowa, 1963
		12.6	1	1.26	+0.315	1.18	
		13.5	1	1.35	+0.55	1.16	
Undisturbed San Francisco Bay Mud	w_L = 88% PI = 45%	N.C.	-	~0.50	+0.20 ±.04	0.60 ±.02	Seed, Noorany and Smith (1964)
Undisturbed London Clay	w_L = 95% PI = 65%	Very O.C.	1	2.0	+0.30	1.70	Skempton, 1961

* O.C.R. = Overconsolidation ratio

Table 2.4 Effect of Perfect Sampling on Undrained Strength Behavior of Normally Consolidated Remolded Boston Blue Clay

Stresses in kg/cm^2 $\qquad$ W_L = 33 ± 3 %, $\quad$ PI = 15 ± 2 % $\qquad$ Batch $\bar{\sigma}_c$ = 1.5 kg/cm^2

Test No.	w_i %	w_f %	Consolidation $\bar{\sigma}_{1c}$	$\bar{\sigma}_{3c}$	K	Perfect Sampling $\bar{\sigma}_{ps}$	Au	$\bar{\sigma}_{ps}/\bar{\sigma}_{1c}$	At $(\sigma_1-\sigma_3)$ max.(1) ε %	$(\sigma_1-\sigma_3)$	$\bar{\sigma}_3$	q	$\bar{p}$	$\bar{\sigma}_1/\bar{\sigma}_3$	$S_u/\bar{\sigma}_{1c}$	A_f(2)	At $(\bar{\sigma}_1/\bar{\sigma}_3)$ max. ε %	q	$\bar{p}$	Pore Fluid	Batch No.	Tested By Year
$\overline{CAU}$-1	31.2	25.9	4.10	2.16	0.528	—	—	—	0.27	2.695	1.69	1.35	3.04	2.59	0.329	0.62	9.3	0.92	1.72	16 g/l NaCl	S-5	J.V. 1963
CA-$\overline{UU}$-1	31.3	26.5	4.08	2.16	0.53	2.52	0.187	0.618	0.65	2.24	1.44	1.12	2.56	2.56	0.275	0.482	9.7	0.78^5	1.43^5	//	S-5	J.V. 1963
$\overline{CAU}$-2	29.7	23.3	6.10	3.27	0.536	—	—	—	0.31	4.00	2.64	2.00	4.64	2.52	0.328	0.535	8.4	1.36^5	2.37^5	//	S-6	J.V. 1963
CA-$\overline{UU}$-2	31.0	24.9	6.15	3.24	0.528	3.96	0.247	0.645	0.73	3.48	2.20	1.74	3.94	2.53	0.283	0.505	8.2	1.32	2.39	//	S-5	J.V. 1963
$\overline{CAU}$-3	27.2	23.7	2.92	1.59	0.545	—	—	—	0.3	1.96	1.27	0.98	2.25	2.55	0.336	0.51	5.9	0.765	1.45	Fresh Water	F-1	WAB 1961
$\overline{CAU}$-1	28.1	25.0	3.10	1.66	0.535	—	—	—	0.30	1.93^5	1.32	0.97	2.29	2.46	0.312	0.59	8.3	0.73	1.32	//	F-2	WAB 1962
CA-$\overline{UU}$-1	27.8	24.6	3.10	1.66	0.535	1.92	0.18	0.62	0.74	2.02	1.23	1.01	2.24	2.64	0.328	0.34	5.3	0.86	1.57	//	F-2	WAB 1962
$\overline{CAU}$-2a	27.2	21.1	6.00	3.18	0.530	—	—	—	0.5	3.99	2.46	2.00	4.[illegible]6	2.62	0.333	0.615	5.6	1.77	3.34	//	F-1	WAB 1961
$\overline{CAU}$-2b	28.0	22.3	6.10	3.32	0.545	—	—	—	0.55	3.99	2.47	2.00	4.47	2.62	0.328	0.70	7.7	1.59	2.85	//	F-2	WAB 1962
CA-$\overline{UU}$-2	27.7	22.3	6.10	3.32	0.545	3.66	0.12	0.60	1.1	3.65	2.02	1.82^5	3.85	2.80	0.300	0.45	6.8	1.62	2.94	//	F-2	WAB 1962

(1) All tests are triaxial compression tests

(2) $A_f = (\Delta u - \Delta\sigma_3)/(\Delta\sigma_1 - \Delta\sigma_3)$ starting from $\bar{\sigma}_{ps}$ for the CA-$\overline{UU}$ tests.

Table 2.5 Effect of Perfect Sampling on Undrained Strength Behavior of Normally Consolidated Undisturbed Kawasaki Clays

Stresses in kg/cm²

CLAY	Depth m	$\bar{\sigma}_{vo}$	P.I. %	Test No.	w_i %	w_f %	Consolidation $\bar{\sigma}_{1c}$	$\bar{\sigma}_{3c}$	K	Perfect Sampling $\bar{\sigma}_{ps}$	Au	$\bar{\sigma}_{ps}/\bar{\sigma}_{1c}$	At $(\sigma_1-\sigma_3)$ max.[1] ϵ %	$(\sigma_1-\sigma_3)$	$\bar{\sigma}_3$	q	$\bar{p}$	$\bar{\sigma}_1/\bar{\sigma}_3$	$Su/\bar{\sigma}_{1c}$	A_f[3]	At $(\bar{\sigma}_1/\bar{\sigma}_3)$ max. ϵ %	q	$\bar{p}$	$\bar{\sigma}_1/\bar{\sigma}_3$
I	21	1.6	~25	$\overline{CA}U$-(18-4)-1	57.6	49.8	3.20	1.50	0.47	—	—	—	0.8	2.42	1.05	1.21	2.26	3.30	0.378	0.63	5.7	1.12	1.76	4.50
				CA-$\overline{UU}$-(18-4)-1	61.5	53.0	3.20	1.50	0.47	1.61	0.065	0.505	3.0	2.14	0.80	1.07	1.87	3.67	0.334	0.38	~ 7	~1.00	~1.60	~4.33
I	25	1.9	~35	$\overline{CA}U$-4-1a	79.0	71.6	1.79	1.00	0.56	—	—	—	1.0	1.60	0.62	0.80	1.42	3.58	0.446	0.47	3	0.60	1.27	4.38
				CA-$\overline{UU}$-4-1[2]	73.6	64.0	1.75	1.00	0.57	?	?	?	10.3	1.37	—	0.685	—	—	0.39	—	—	—	—	—
II	35	2.6	~46	$\overline{CA}U$-(18-8)-1	88.3	71.3	5.40	2.50	0.465	—	—	—	0.7	4.05	1.89	2.03	3.92	3.14	0.375	0.53	6.7	1.83	2.83	4.65
				CA-$\overline{UU}$-(18-8)-1	88.2	71.0	5.40	2.50	0.465	3.30	0.275	0.610	1.7	3.85	1.50	1.93	3.43	3.57	0.355	0.48	5.4	1.78	2.88	4.25
III	45	3.5	~16	$\overline{CA}U$-(18-11)-1	42.1	36.2	7.41	3.50	0.471	—	—	—	0.45	6.32	2.76	3.16	5.92	3.29	0.426	0.31	5.7	2.71	4.51	4.01
				CA-$\overline{UU}$-(18-11)-1	40.7	34.6	7.55	3.50	0.465	3.97	0.115	0.525	1.2	5.75	2.36	2.88	5.24	3.44	0.382	0.28	8.2	2.64	4.42	3.97

(1) All tests are triaxial compression tests

(2) Pore pressure measurements were no good

(3) $A_f = (\Delta u - \Delta\sigma_3)/(\Delta\sigma_1 - \Delta\sigma_3)$ starting from $\bar{\sigma}_{ps}$ for the CA-$\overline{UU}$ tests

Table 2.6 Effect of Perfect Sampling on Undrained Strength Behavior of Normally Consolidated Clays

All Stresses in kg/cm²

Clay	Type of Test	No. of Tests	At Consolidation		Perfect Sampling		At $(\sigma_1 - \sigma_3)$ max.				Atterberg Limits	Reference
			$\bar{\sigma}_{1c}$	K	A_u	$\bar{\sigma}_{ps}/\bar{\sigma}_{1c}$	ε %	$s_u/\bar{\sigma}_{1c}$	A_f	$\bar{\phi}^{\circ}$ (1)		
Remolded Boston Blue Clay (Fresh and Salt Water)	$\overline{CAU}$	6	2.9 - 6.1	0.54 ±.01	—	—	0.4 ±.1	0.325 ±.01	0.60 ±.10	26.0 (25.0-26.6)	W_L = 33 % P.I. = 14 %	Table 2.4
	CA-$\overline{UU}$	4	3.1 - 6.1	0.54 ±.01	0.18 ±.06	0.62 ±.05	0.8 (.65-1.1)	0.265 (.275-.325)	0.44 (.34-.51)	26.8 (26.0-28.3)		
Undisturbed Kawasaki Clays	$\overline{CAU}$	4	1.8 - 7.4	0.49 (.47-.56)	—	—	0.75 (.45-1.0)	0.405 (.38-.446)	0.49 (.31-.62)	32.5 (31.0-34.3)	P.I = 16-46%	Ladd and Lambe (1963) and Table 2.5
	CA-$\overline{UU}$	4	1.8 - 7.6	0.49 (.46-.57)	0.15 (2) (.07-.29)	0.545 (2) (.50-.61)	4.6 (1.2-10.5)	0.365 ±.03	0.38 (2) ±.10	34.2 (2) (33.3-34.9)		
(3) Undisturbed San Francisco Bay Mud	$\overline{CAU}$	5	0.8 - 1.6	0.50	—	—	4.7 (4.5-5)	0.39 (.37-.405)	0.87 (.81-.99)	~32 (4)	W_L = 88 % P.I. = 45 %	Seed, et al (1964)
	CA-$\overline{UU}$	5	0.8 - 1.6	0.50	~0.13 (5)	~0.56	5.5 (4.5-6.5)	0.365 (.35-.385)	0.46 (.43-.48)	~33.5 (4)		
Remolded Weald Clay	$\overline{CAU}$	9	2 - 12	0.60 (.58-.64)	—	—	6.3 (5-9)	0.265 (.25-.275)	2.05 (1.6-2.9)	25.4 ±.7	W_L = 46 % P.I. = 24 %	Skempton and Sowa (1963)
	CA-$\overline{UU}$	4	2 - 11.7	0.61 (.59-.64)	-0.033 ±.02	0.58 (.57-.61)	7.3 (5.6-9)	0.260 (.25-.27)	0.48 (.45-.53)	26.0 ±1.2		

(1) For δ = 0 (2) Ave. of 3 tests (3) Values read from figures and are therefore approximate

(4) For δ = 0.035 (5) Text quoted ave. Au = 0.20

Table 2.7 **Effect of Intermediate Principal Stress on Undrained Strength Parameters of N. C. Remolded Sault Ste Marie Clay**

(Wu, Loh and Malvern, 1963)

Test Series	Type of Test (4)	Ave. Values for $\bar{\sigma}_c$ = 2.6 to 4.0 kg/cm²					
		$S_u/\bar{\sigma}_c$	A_f	$\bar{\Phi}$ (1)	ε_f % (2)	$\Delta u/\bar{\sigma}_c$ (3)	$\bar{\sigma}_{zf}/\bar{p}_f$
CI	Triaxial Compression $\sigma_z > \sigma_r = \sigma_\theta$	0.465	0.63	32	14	0.58	0.475
CIa	Hollow Cylinder Compression $\sigma_z > \sigma_r = \sigma_\theta$	0.42	0.78	33.5	13	0.655	0.45
—	Ave. CI and CIa	0.44	0.70	32.5	13.5	0.62	0.46
I 2	Hollow Cylinder $\sigma_z > \sigma_\theta > \sigma_r$ (incr. Po, then incr. σ_z to failure)	0.445	0.685	32.5	7	0.61	0.865
I_1 and I_3	Hollow Cylinder $\sigma_\theta > \sigma_z > \sigma_r$ (incr. σ_z then incr. P_θ to failure)	0.42	0.62	28.5	—	0.52	1.13
E1	Triaxial Extension $\sigma_r = \sigma_\theta > \sigma_z$	0.33	1.10	33	10	0.73	1.55

(1) Assuming $\delta = 0$

(2) Axial strain at failure

(3) For $\Delta\sigma_3 = 0$

(4) σ_z = axial stress

σ_r = average radial stress

σ_θ = tangential stress

P_o = outside cylinder pressure

Table 2.8 Summary of Effect of Intermediate Principal Stress on Undrained Strength Parameters of Saturated Clays

Clay [1]		Type of Test [2]	Tests with $\sigma_2 \approx (\sigma_1+\sigma_3)/2$ versus Compression Tests			Extension Tests vs. Compression Tests			Reference
			S_u	A_f	$\bar{\Phi}$	S_u	A_f	$\bar{\Phi}$ [3]	
Undisturbed Boston Blue Clay	N.C.	Triaxial				10-20% decrease	Increased	Same	Taylor (1955)
	O.C.	Triaxial				10-20% decrease	Increased	Slight increase	
Three N.C. Undisturbed Clays		Triaxial				20-25% decrease	Increased	Same	Hirschfeld (1958)
Remolded Weald Clay	N.C.	Triaxial				15% decrease	Increased by 0.28	Same	Parry (1960)
	O.C.	Triaxial				15% decrease	Increased by 0.18-0.20	Slight increase	
Remolded N.C. Sault Ste Marie Clay		Triaxial				30% decrease	Increased by 0.47	Same	Wu, et al (1963)
		Hollow Cylinder	Same	Slight decrease	Slight decrease	22% decrease	Increased by 0.32	Same	
Remolded N.C. Kaolinite		Hollow Cylinder	Same or slight increase	Increased by 0.17-0.32	Increased by 6°	20-30% decrease	Increased by 0.5-0.6	Increased by 7°	Broms, et al (1964)
Remolded N.C. Weald Clay		Plane strain vs Triaxial (K_o)	8% increase	Decreased by 0.52	Increased by 1.2°				Wade (1963)

(1) N.C. = normally consolidated
O.C. = overconsolidated
(2) Isotropic consolidation unless otherwise stated
(3) For o.c. clays, "increase" refers to a higher effective stress envelope.

F

Fig. 2.1. Stress Difference and Pore Pressure vs Obliquity as a Function of K and A

(For compression tests, $\Delta\sigma_2 = \Delta\sigma_3 = 0$; $\bar{\phi} = 30°$)

$$\text{Eq. 2.2} \cdots \quad \frac{(\sigma_1 - \sigma_3)}{\bar{\sigma}_{1c}} = \left[\frac{\bar{\sigma}_1}{\bar{\sigma}_3} - 1\right]\left[1 - \left(\frac{\Delta u}{\bar{\sigma}_{1c}} + 1 - K\right)\right]$$

$$K = \frac{\bar{\sigma}_{3c}}{\bar{\sigma}_{1c}} \; ; \quad K_o = 1 - \sin\bar{\phi} = \frac{1}{2}$$

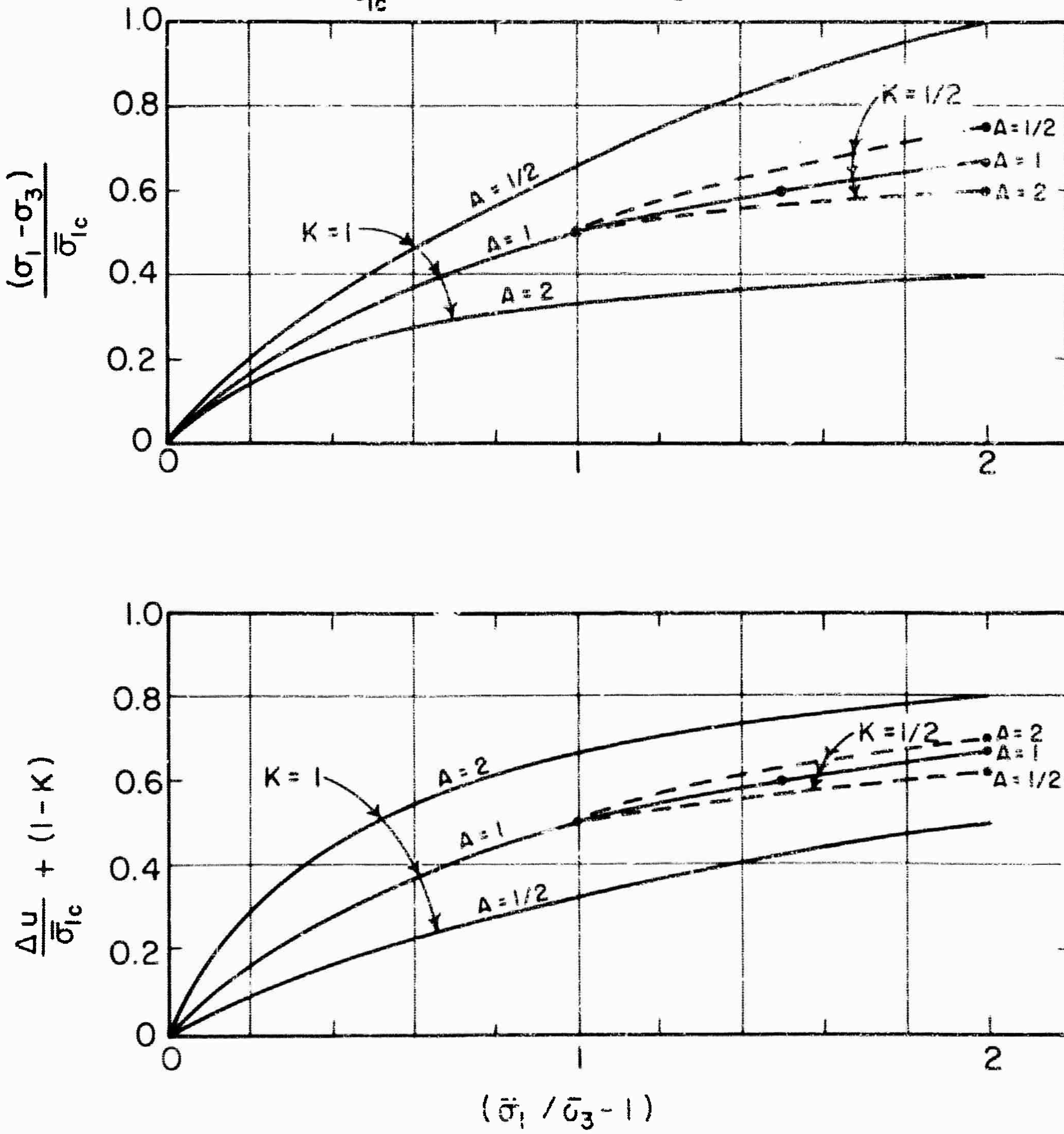


Fig. 2.2. Effect of Anisotropic Consolidation on Undrained Strength

N.C. Remolded Boston Blue Clay

(Consolidated from a Fresh Water Slurry)

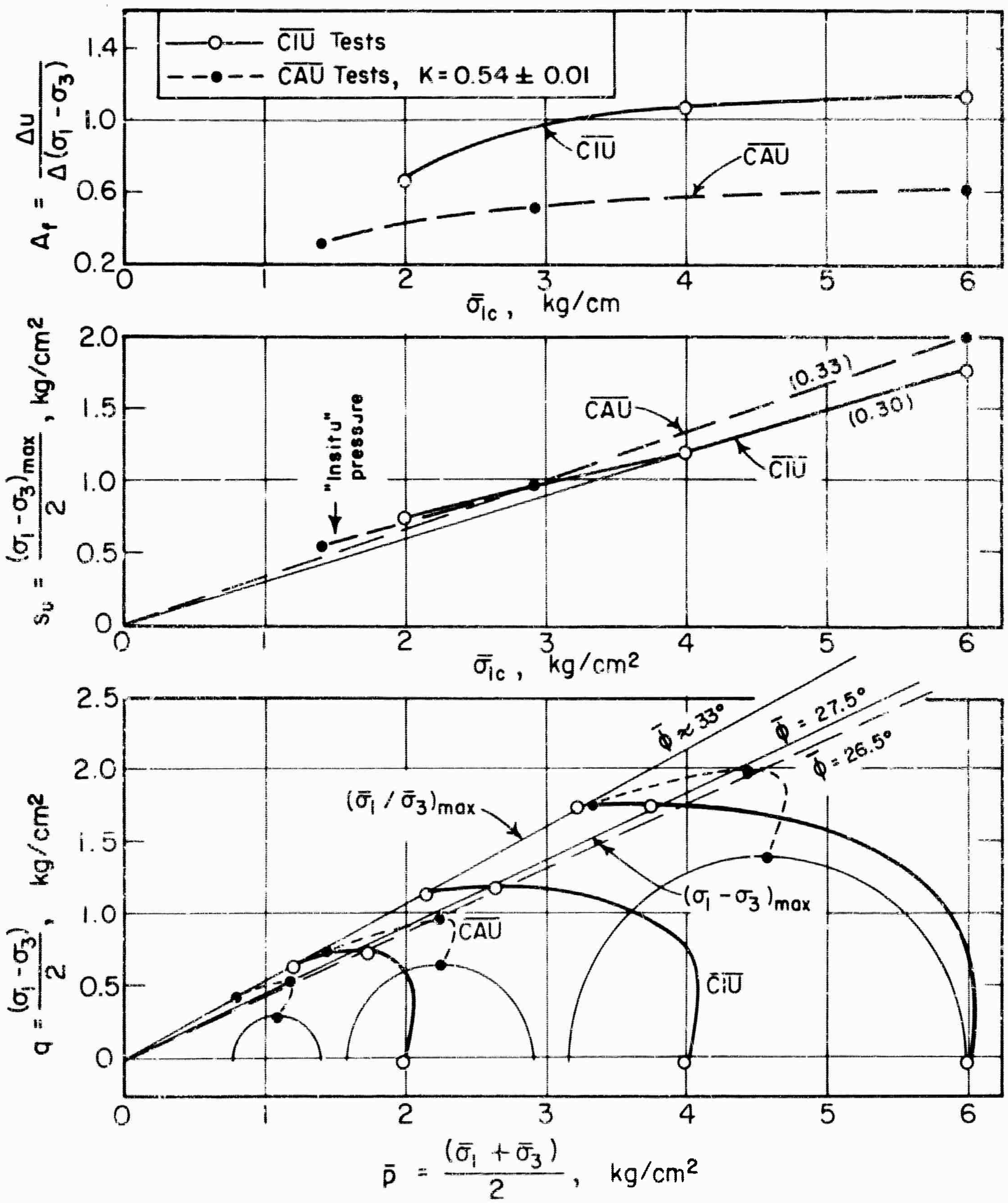


Fig. 2.3. Effect of Anisotropic Consolidation on Undrained Strength

N.C. Remolded Vicksburg Buckshot Clay

(Consolidated from a slurry)

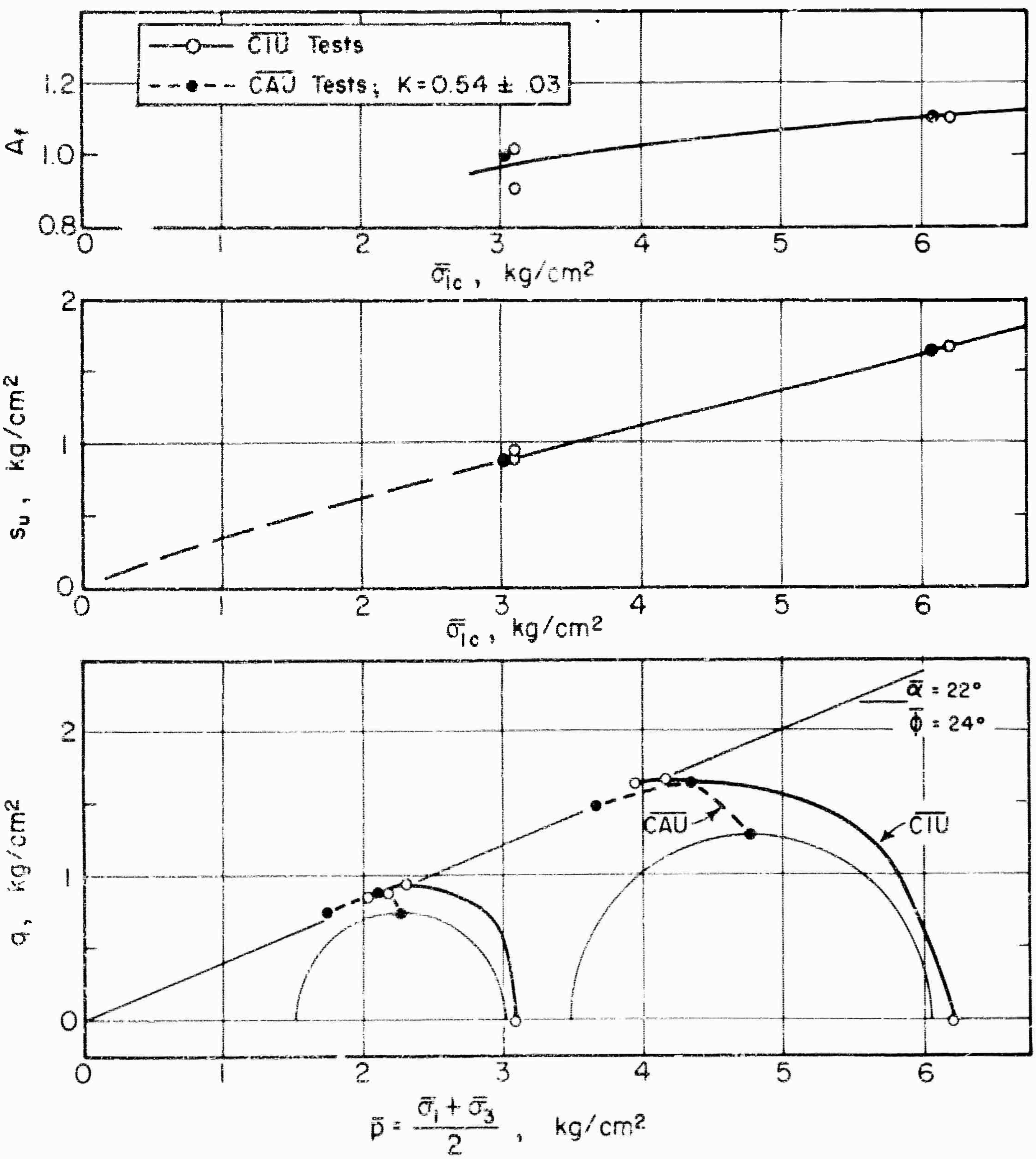


Fig. 2.4. Effect of Anisotropic Consolidation on Undrained Strength

N.C. Undisturbed Kawaski Clays

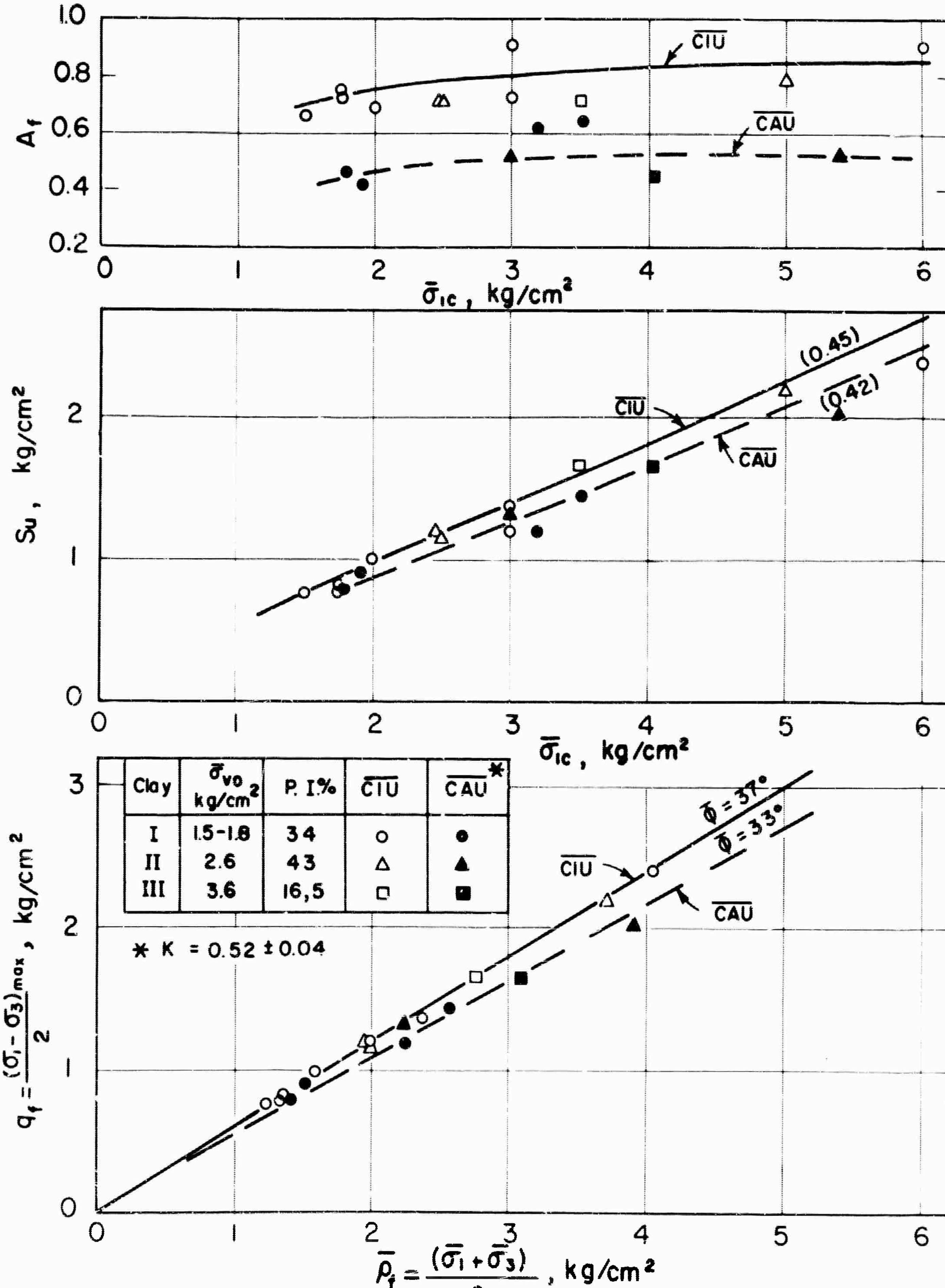


Fig. 2.5. Effect of Anisotropic Consolidation on Undrained Strengh

N.C. Undisturbed Brobekkveien, Oslo, Clay (Simons, 1960)

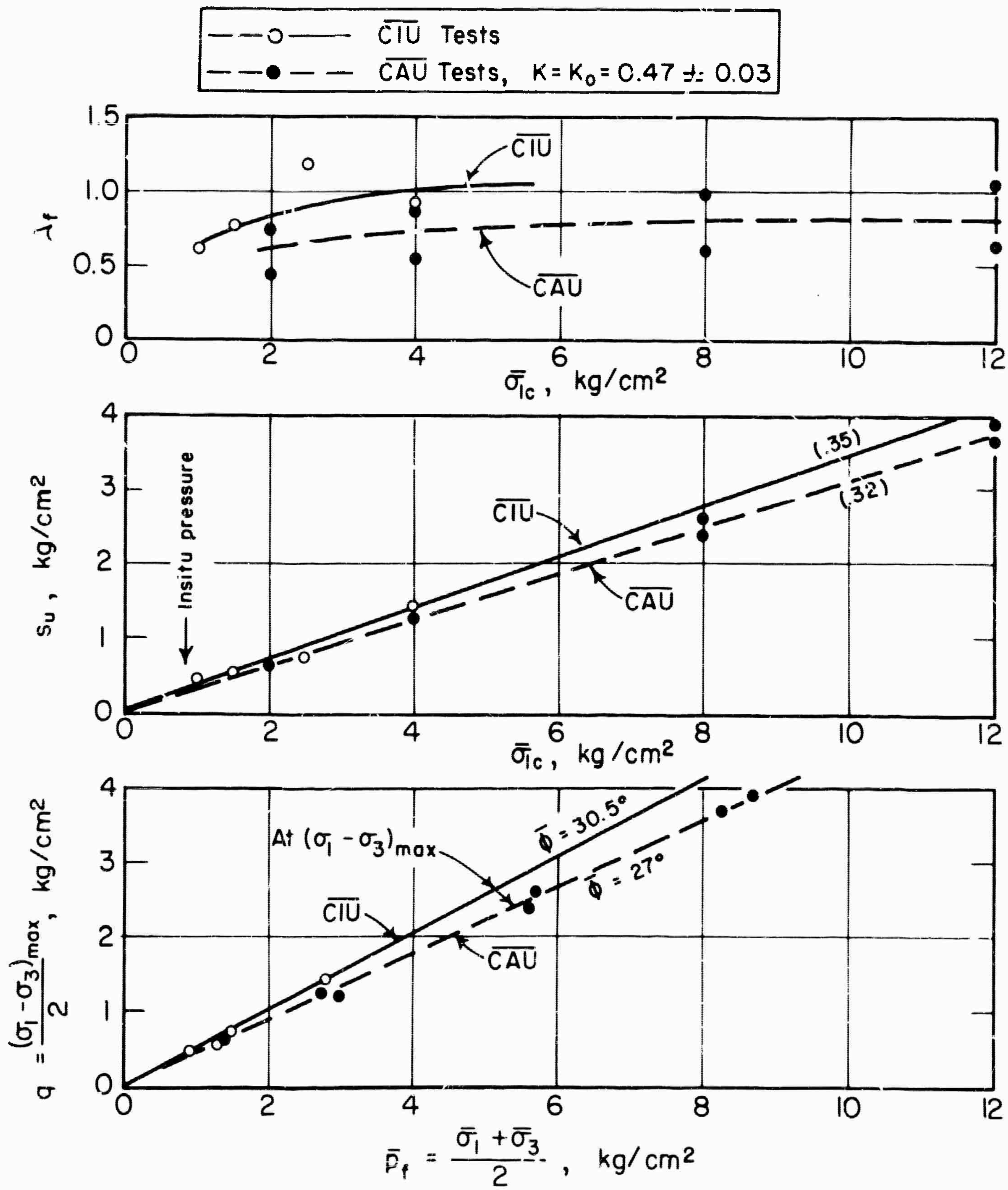


Fig. 2.6. Effect of Anisotropic Consolidation on Undrained Strengh

N.C. Undisturbed Skabo Clay (Landva, 1962)

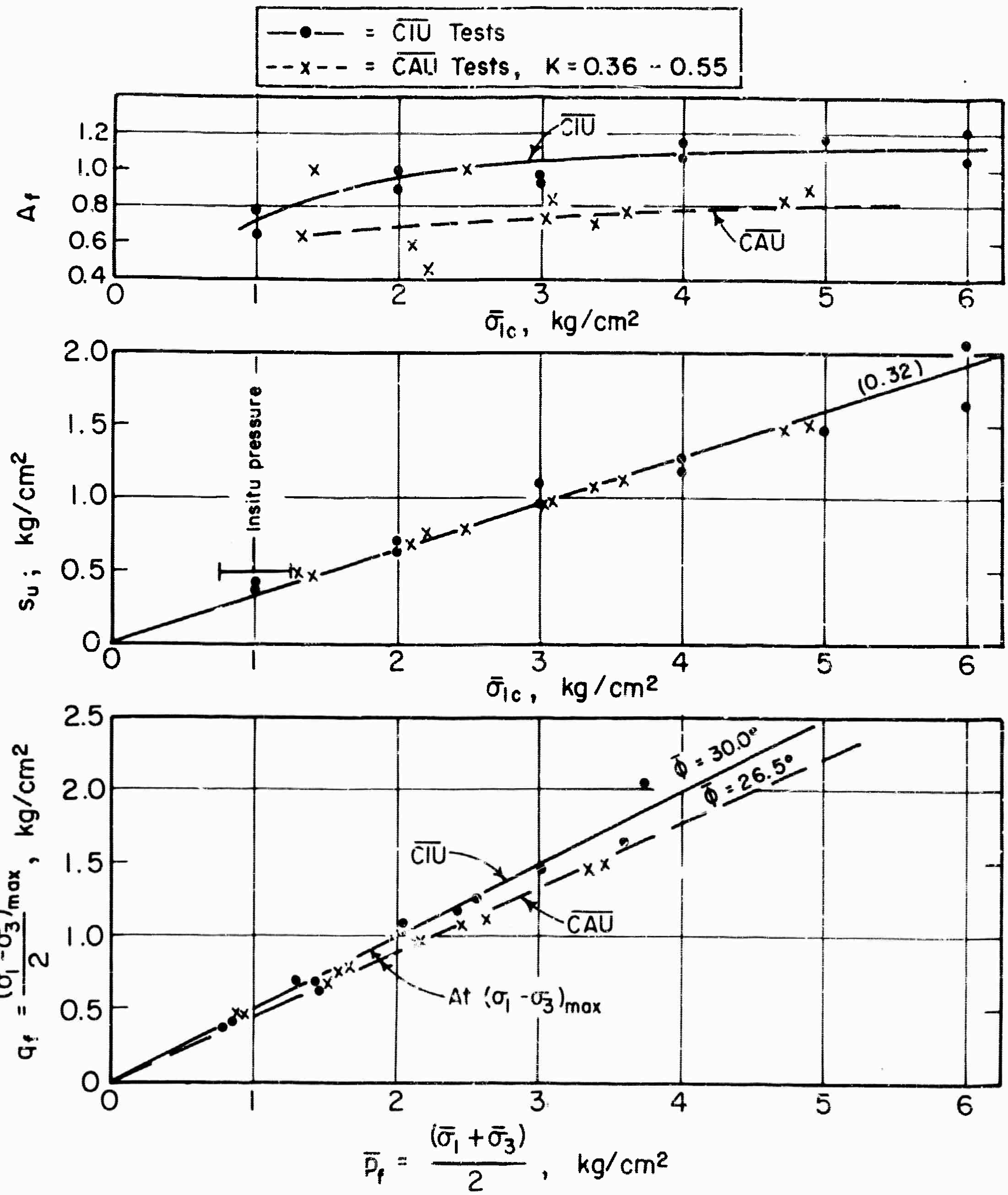


Fig. 2.7. Effect of Anisotropic Consolidation on Undrained Strength

Remolded Weald Clay (Skempton and Sowa, 1963)

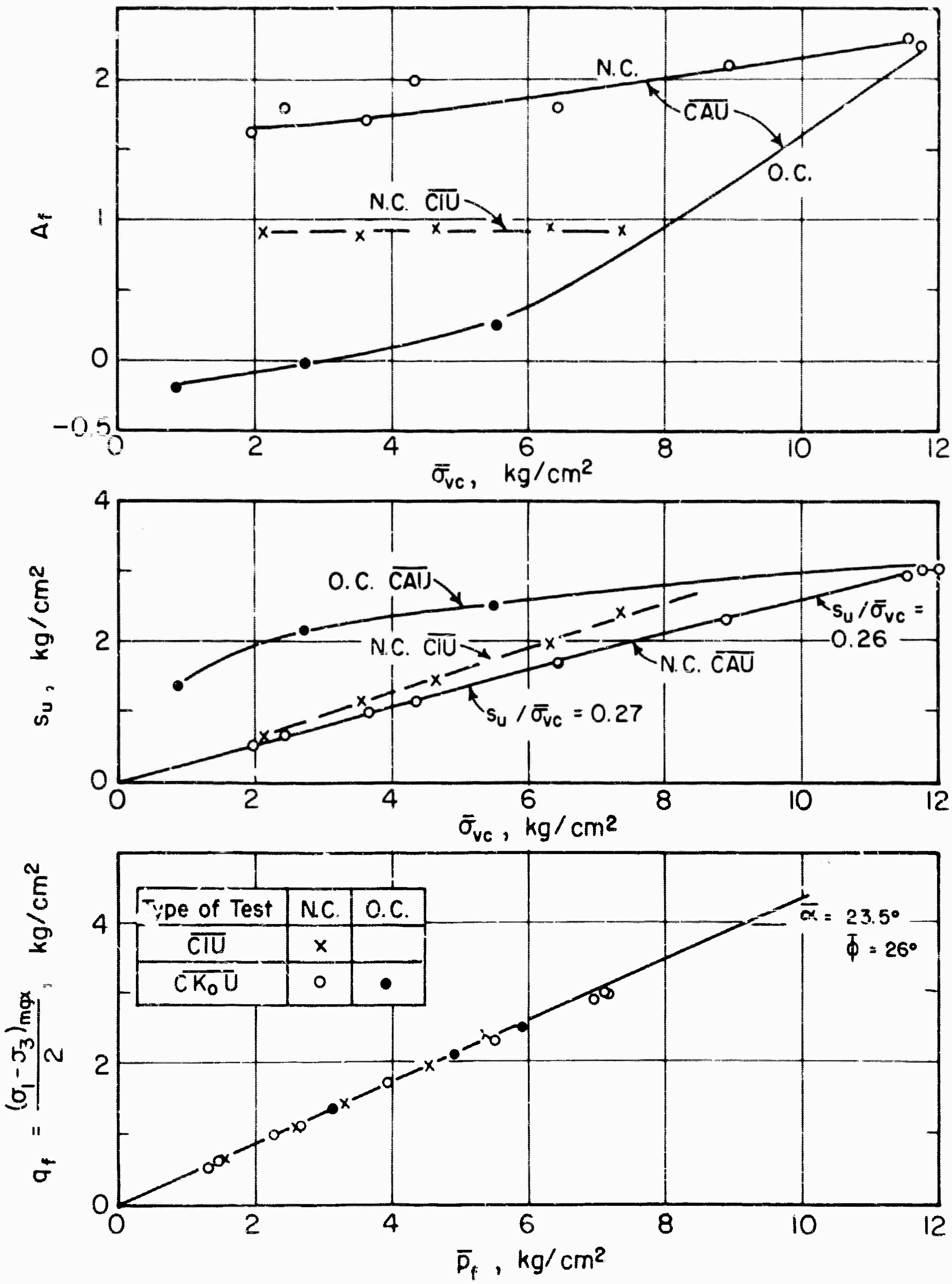
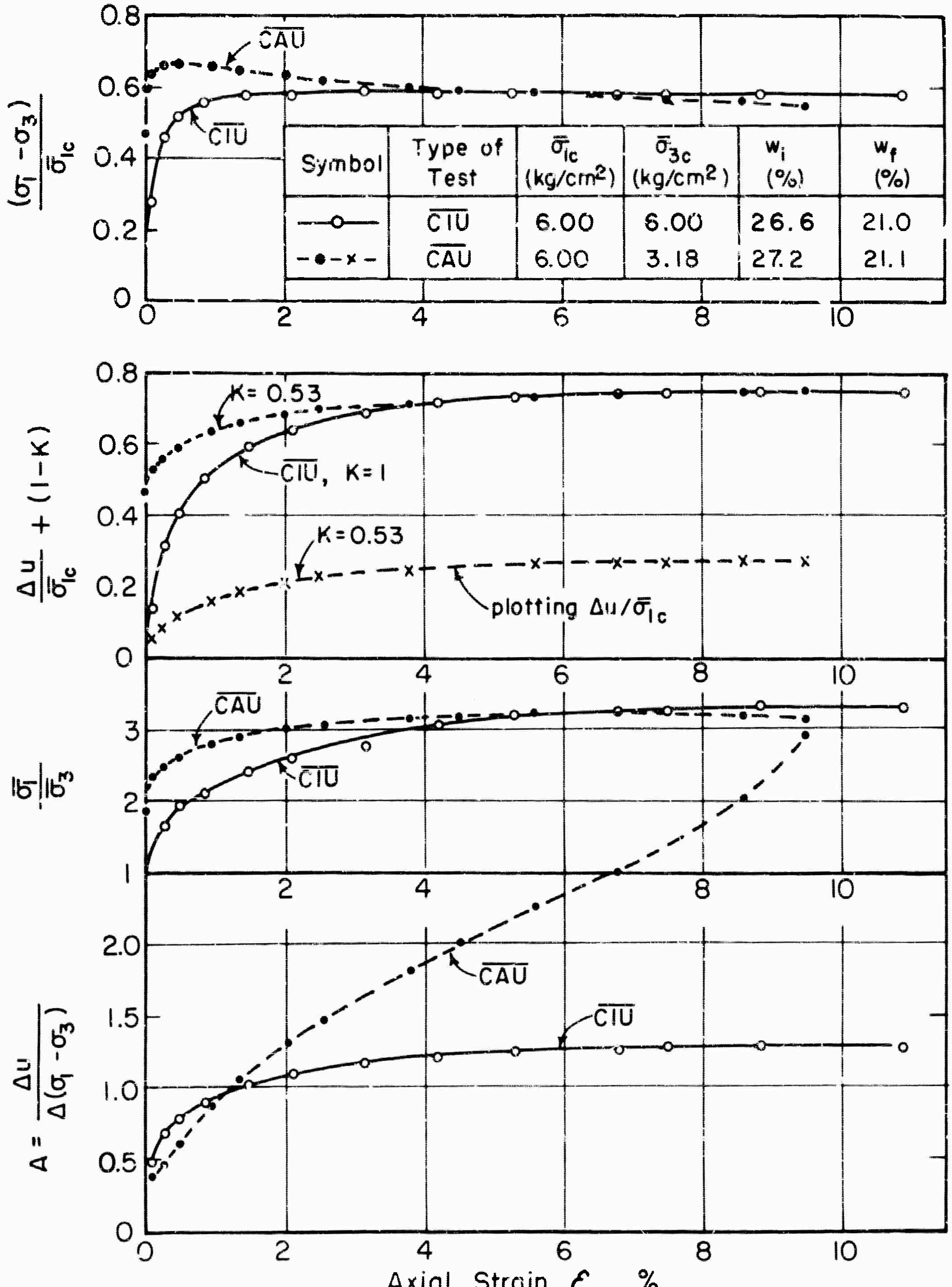


Fig. 2.8. Effect of Anisotropic Consolidation on Stress-Strain Behavior

N.C. Remolded Boston Blue Clay

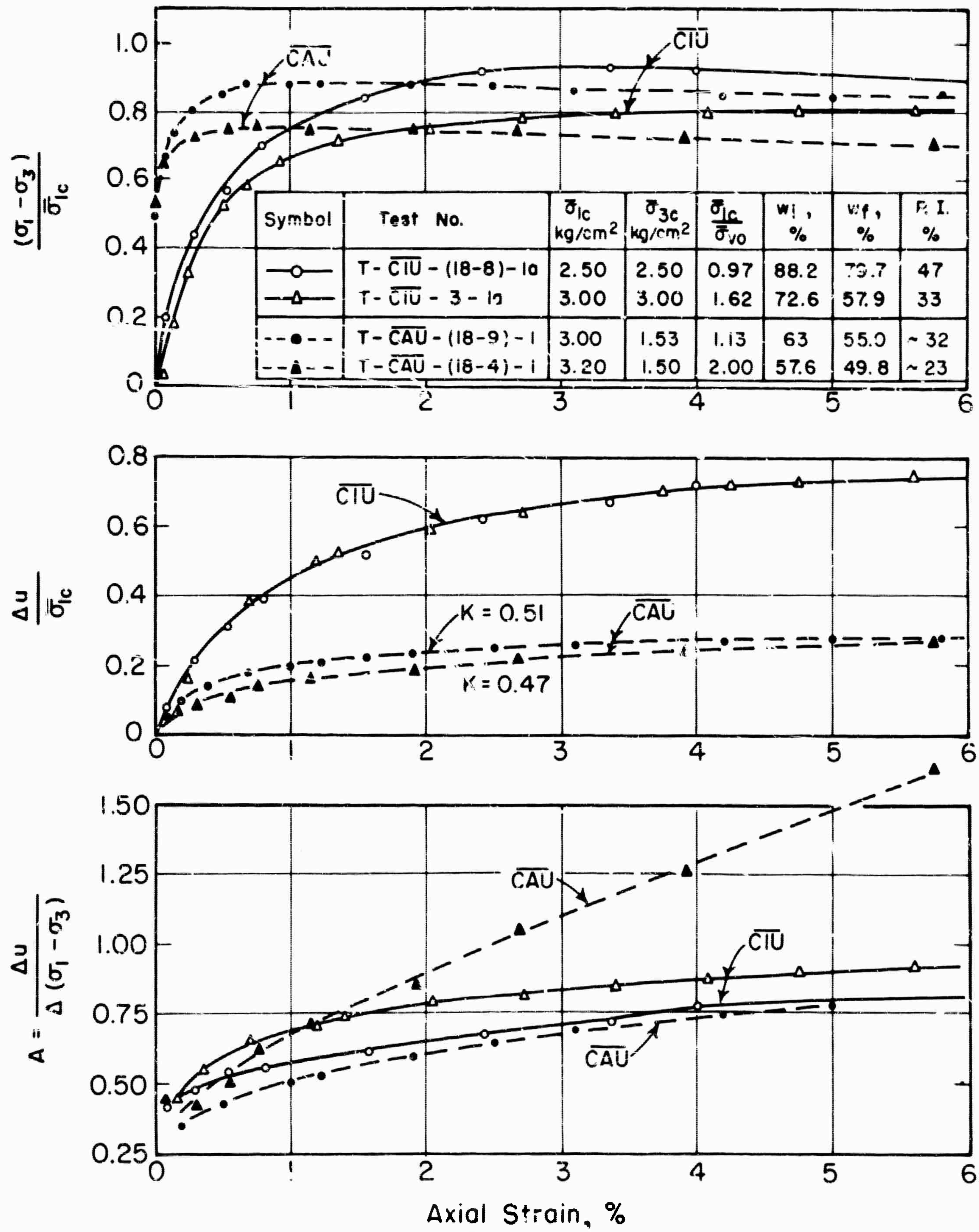
(Consolidated from a Fresh Water Slurry)



Symbol	Type of Test	$\bar{\sigma}_{1c}$ (kg/cm²)	$\bar{\sigma}_{3c}$ (kg/cm²)	w_i (%)	w_f (%)
—o—	$\overline{CIU}$	6.00	6.00	26.6	21.0
-•-x-	$\overline{CAU}$	6.00	3.18	27.2	21.1

Fig. 2.9. Effect of Anisotropic Consolidation on Stress Strain Behavior

N.C. Undisturbed Kawasaki Clays



Symbol	Test No.	$\bar{\sigma}_{1c}$ kg/cm^2	$\bar{\sigma}_{3c}$ kg/cm^2	$\frac{\bar{\sigma}_{1c}}{\bar{\sigma}_{vo}}$	w_i, %	w_f, %	P.I. %
—○—	T-$\overline{CIU}$-(18-8)-1a	2.50	2.50	0.97	88.2	79.7	47
—△—	T-$\overline{CIU}$-3-1a	3.00	3.00	1.62	72.6	57.9	33
--●--	T-$\overline{CAU}$-(18-9)-1	3.00	1.53	1.13	63	55.0	~32
--▲--	T-$\overline{CAU}$-(18-4)-1	3.20	1.50	2.00	57.6	49.8	~23

Fig. 2.10. Stress Difference and Pore Pressure vs Obliquity for $\overline{CIU}$ and $\overline{CAU}$ Tests

N.C. Remolded Boston Blue Clay

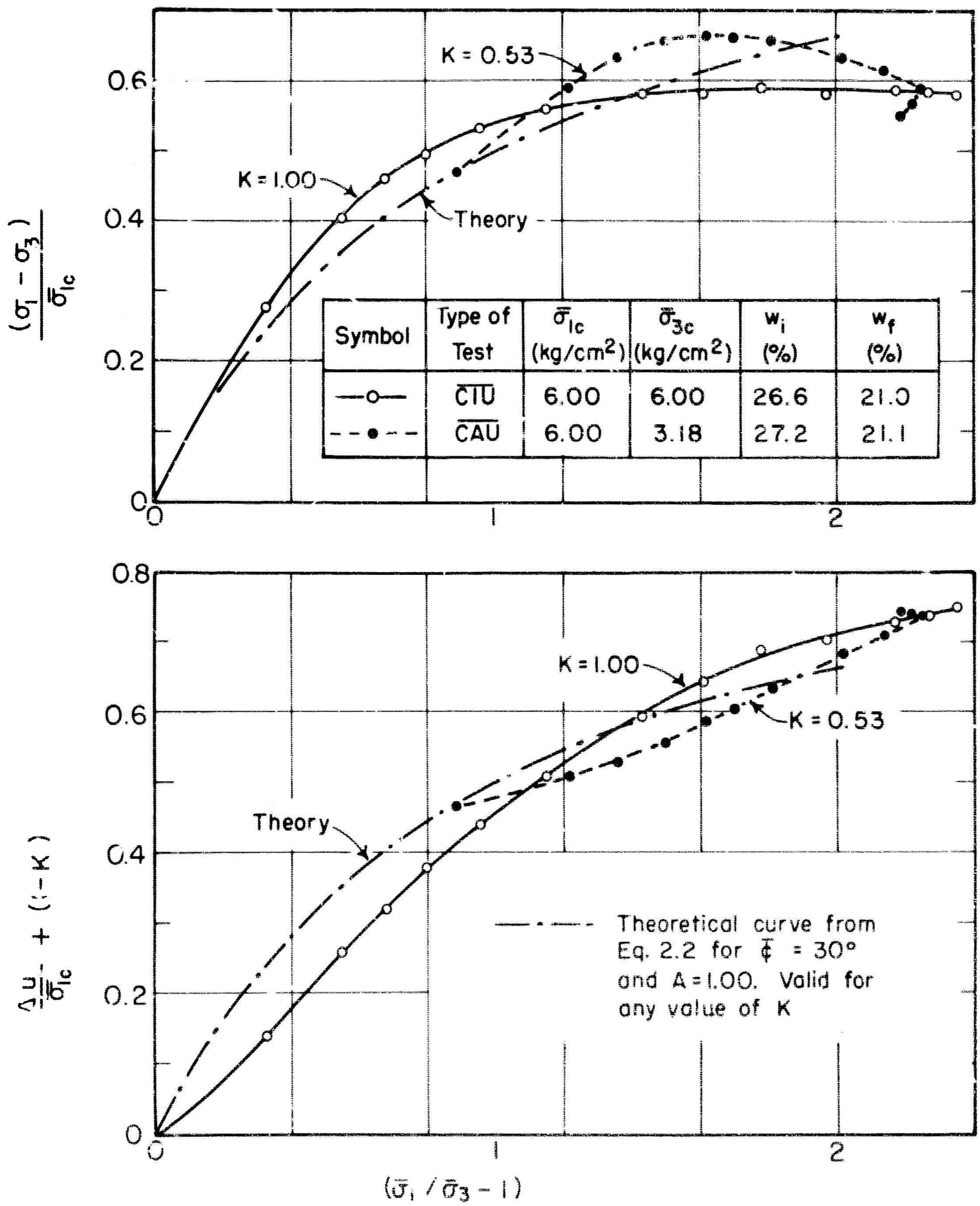
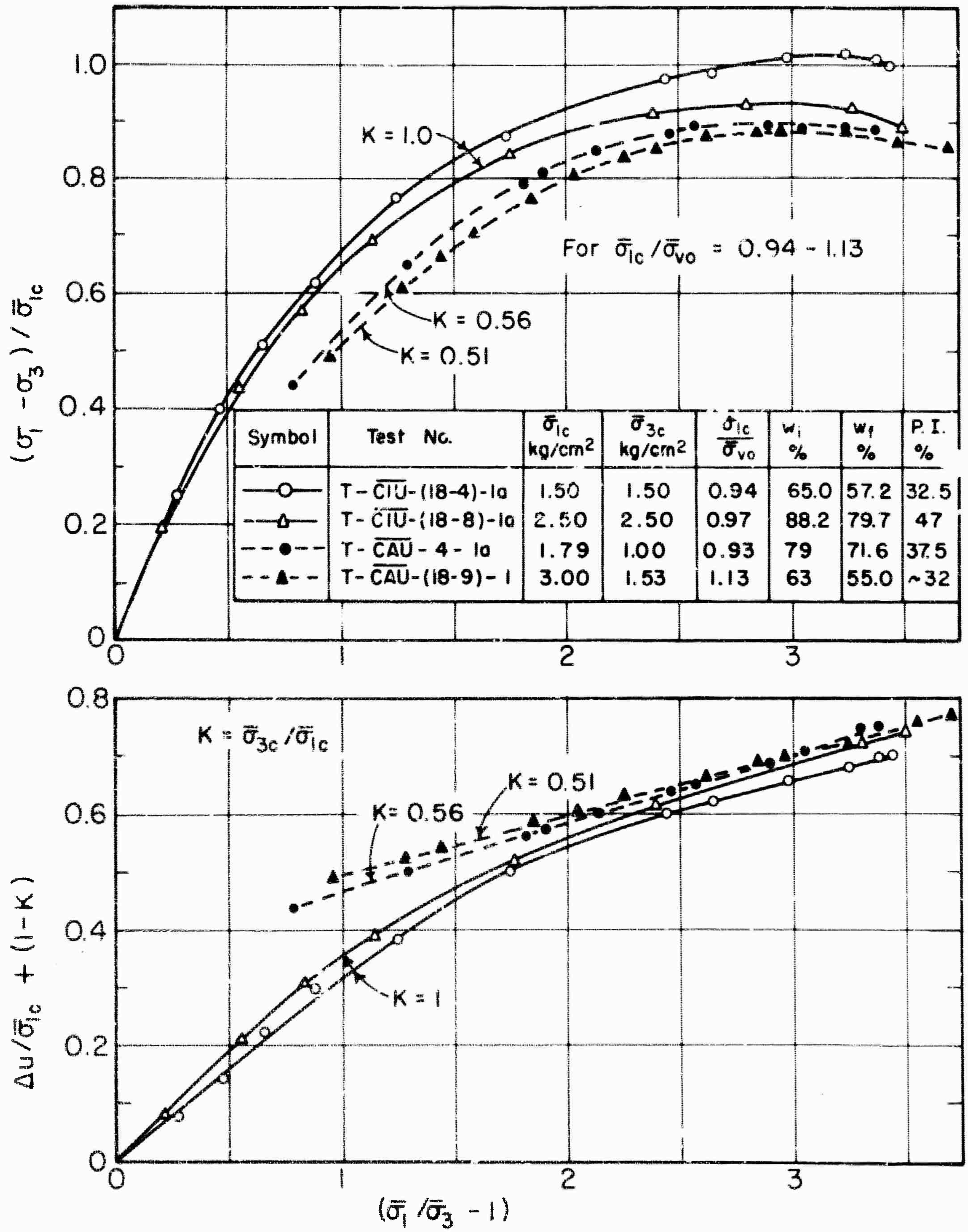


Fig. 2.11. Stress Difference and Pore Pressure vs Obliquity for $\overline{CIU}$ and $\overline{CAU}$ Tests

N.C. Undisturbed Kawasaki Clays



Symbol	Test No.	$\bar{\sigma}_{1c}$ kg/cm²	$\bar{\sigma}_{3c}$ kg/cm²	$\bar{\sigma}_{1c}/\bar{\sigma}_{vo}$	w_i %	w_f %	P. I. %
—o—	T-$\overline{CIU}$-(18-4)-1a	1.50	1.50	0.94	65.0	57.2	32.5
—△—	T-$\overline{CIU}$-(18-8)-1a	2.50	2.50	0.97	88.2	79.7	47
---●---	T-$\overline{CAU}$-4-1a	1.79	1.00	0.93	79	71.6	37.5
--▲--	T-$\overline{CAU}$-(18-9)-1	3.00	1.53	1.13	63	55.0	~32

Fig.2.12 Stress Paths from $\overline{CIU}$ and $\overline{CAU}$ Tests on N.C. Undisturbed Kawasaki Clays

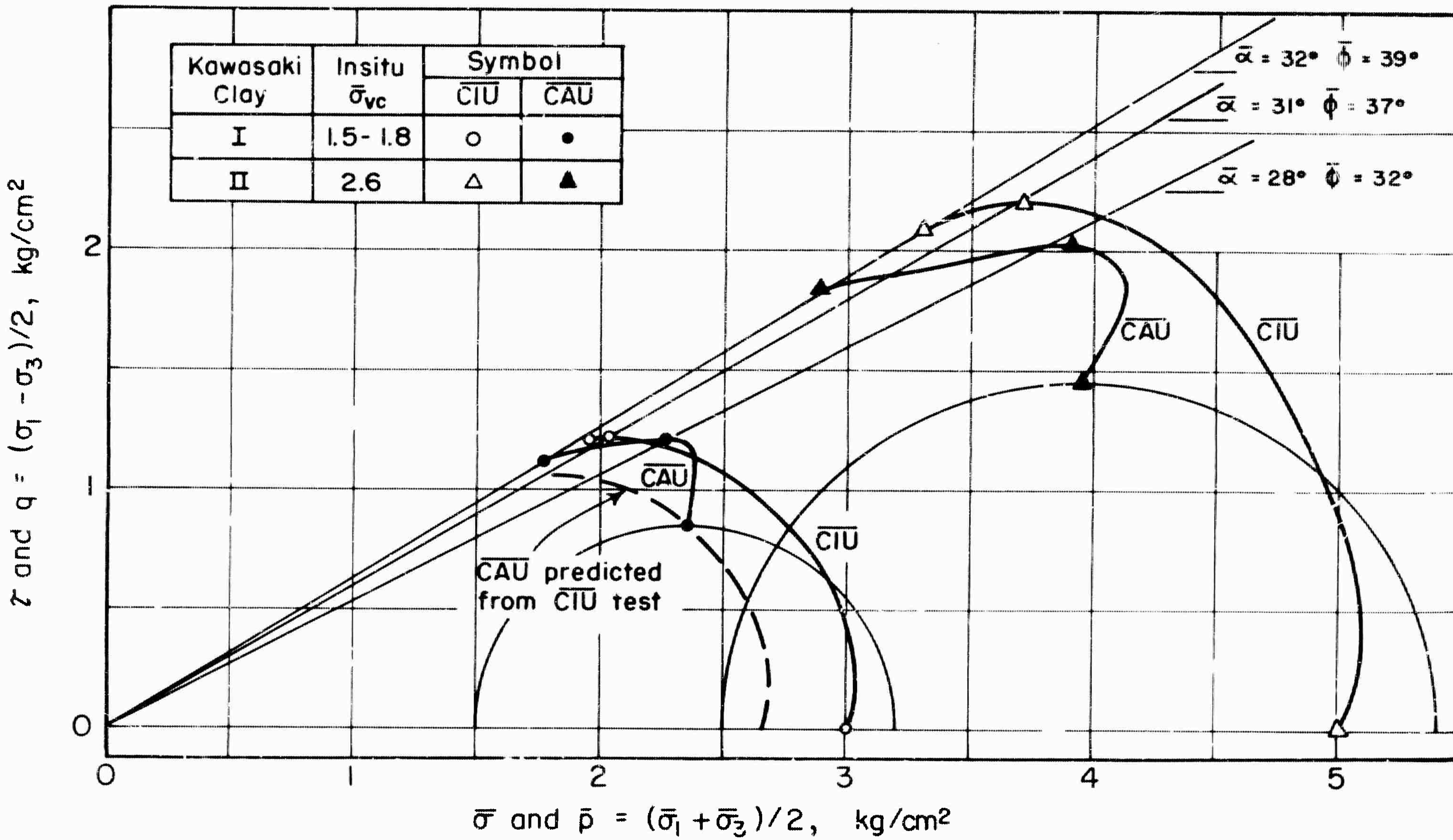


Fig. 2.13. Perfect Sampling of a Normally Consolidated Clay and an Overconsolidated Clay

Note: The one dimensional compression and rebound curves simulate K_o data for the London Clay (Skempton, 1961)

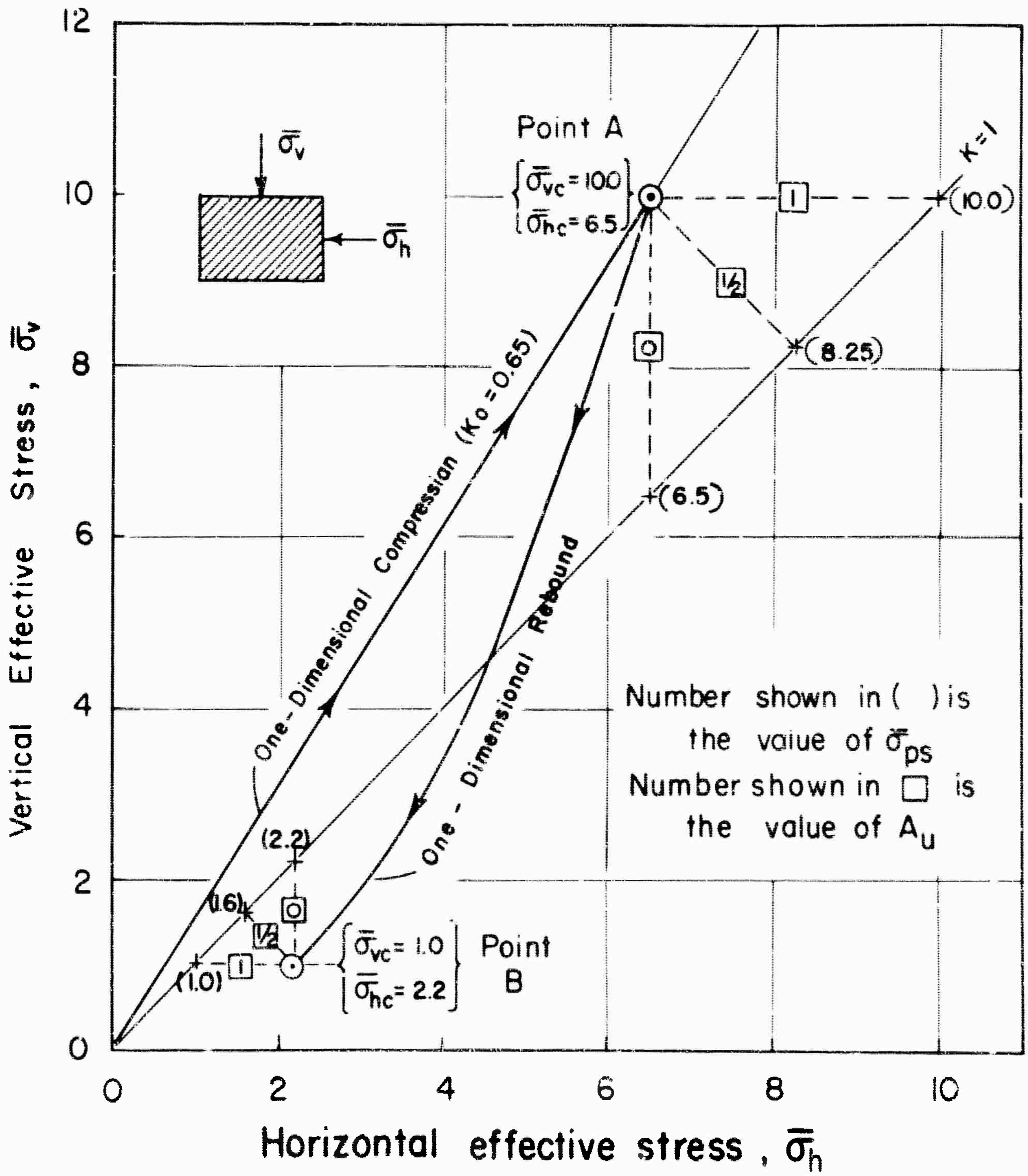
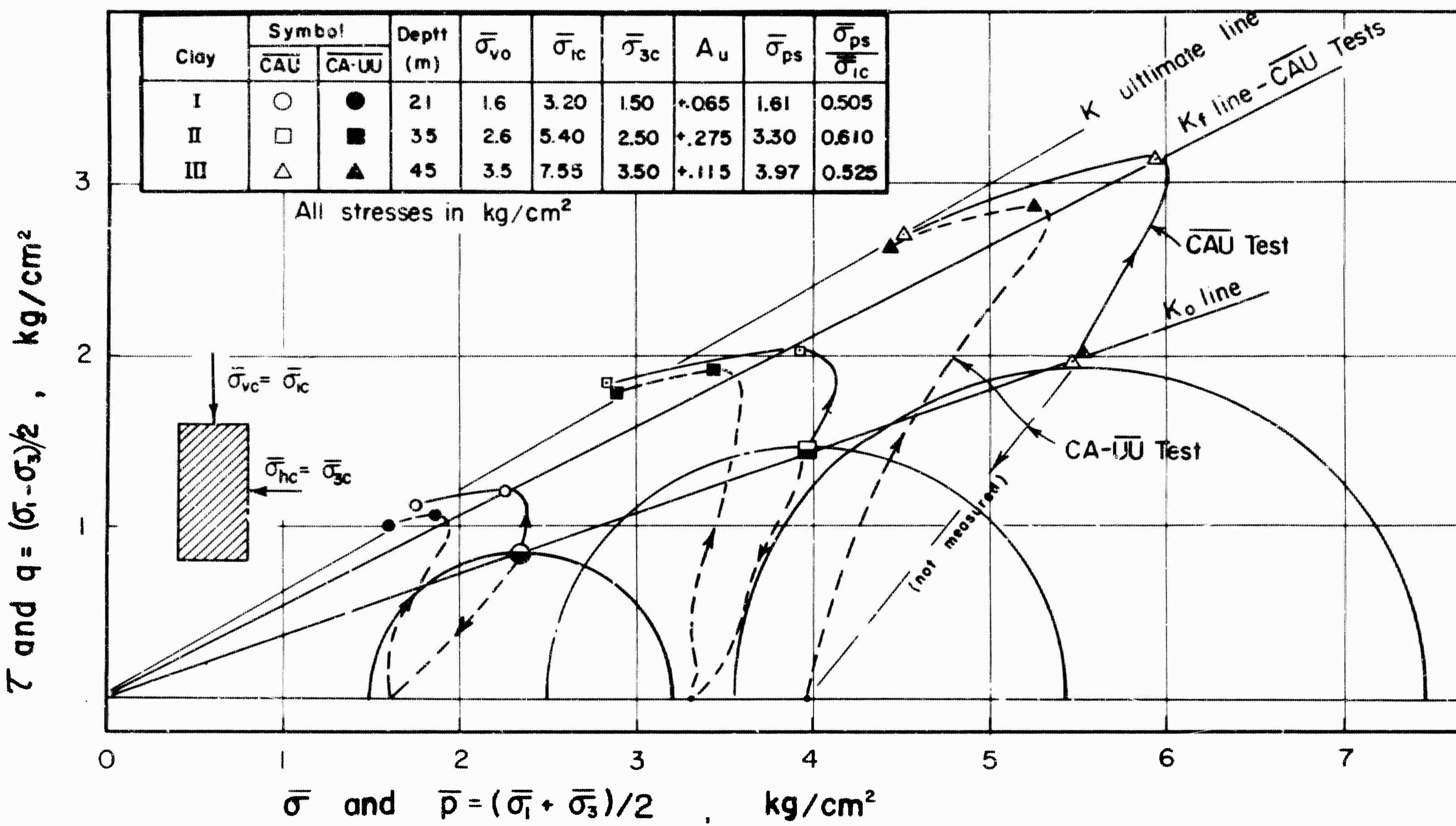


Fig. 2.14. Effect of Perfect Sampling on Stress Paths for Normally Consolidated Kawasaki Clays

(from Ladd and Lambe, 1963)



Clay	Symbol $\overline{CAU}$	Symbol CA-UU	Deptt (m)	$\bar{\sigma}_{vo}$	$\bar{\sigma}_{1c}$	$\bar{\sigma}_{3c}$	A_u	$\bar{\sigma}_{ps}$	$\bar{\sigma}_{ps}/\bar{\sigma}_{1c}$
I	○	●	21	1.6	3.20	1.50	+.065	1.61	0.505
II	□	■	35	2.6	5.40	2.50	+.275	3.30	0.610
III	△	▲	45	3.5	7.55	3.50	+.115	3.97	0.525

All stresses in kg/cm²

Fig. 2.15 Effect of Perfect Sampling on Stress - Strain Behavior of N.C. Undistubed Kawasaki Clay I

Sample No T 18-4

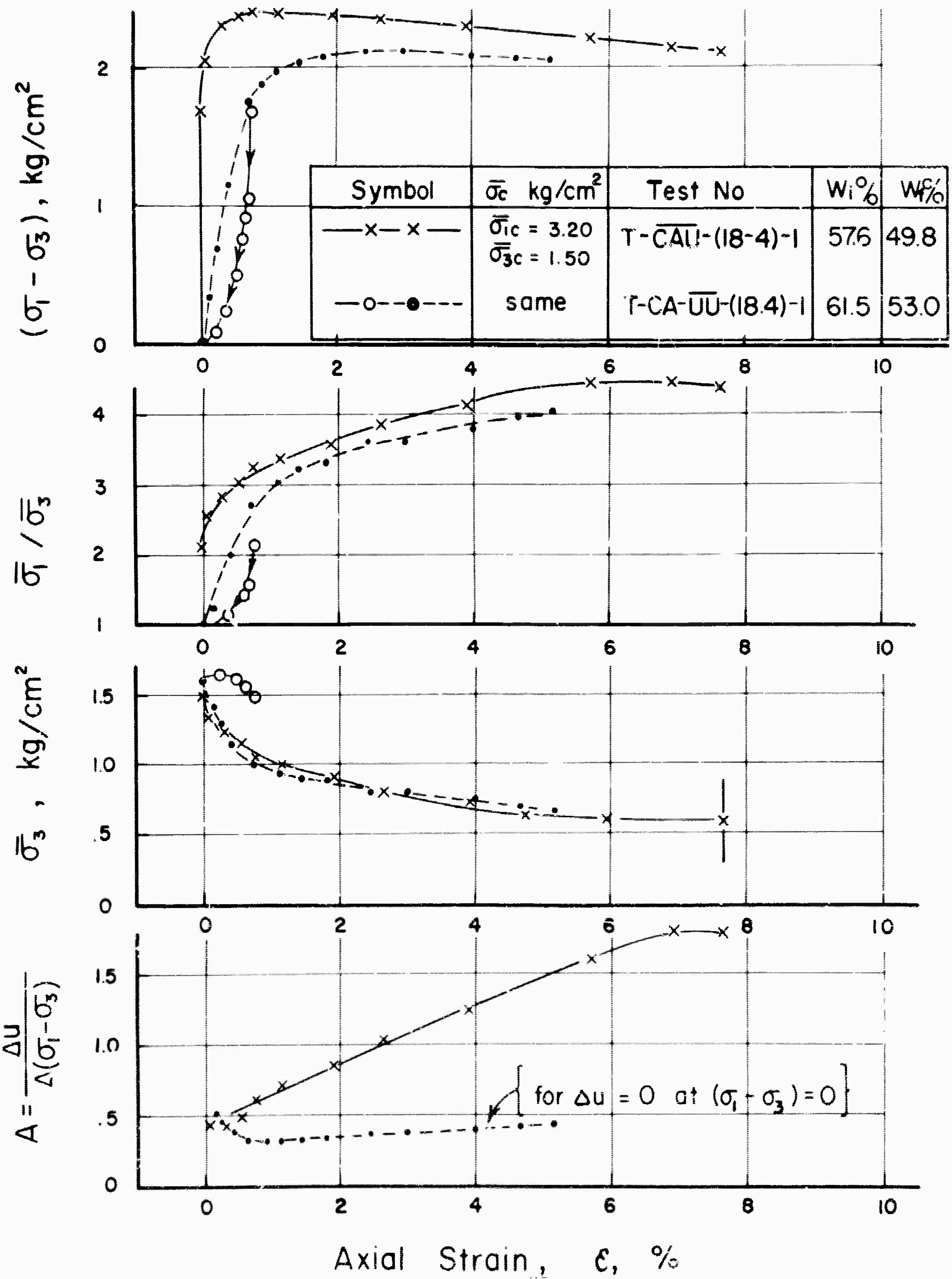


Fig. 2.16 Undrained Strength Parameters for $\overline{CIU}$ C and $\overline{CIU}$ E Tests on Remolded Weald Clay (Parry, 1960)

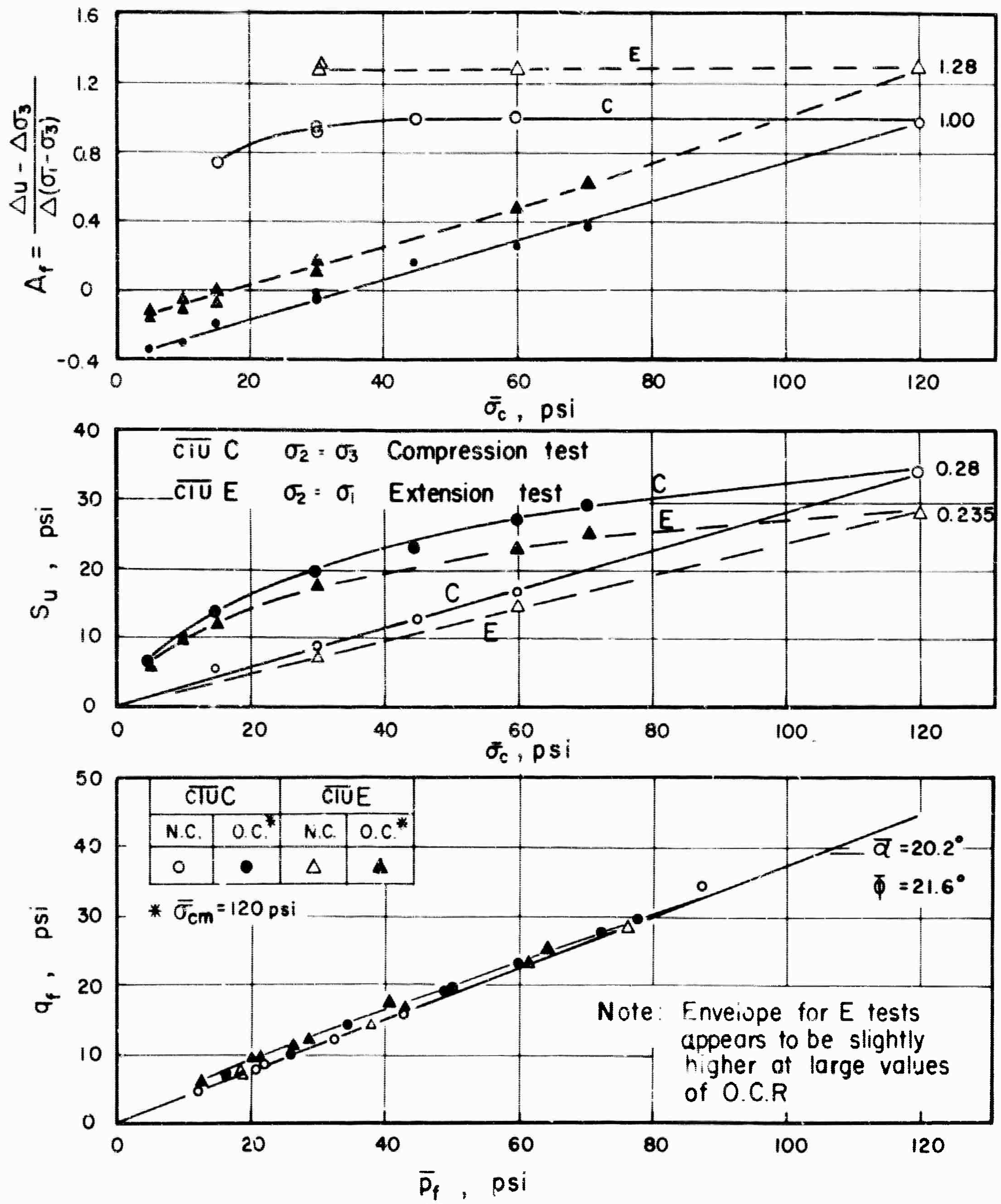


Fig. 2.17. $\overline{CIU}$ Compression and Extension Tests on Remolded Sault Ste Marie Clay

(Wu, Loh & Malvern, 1963)

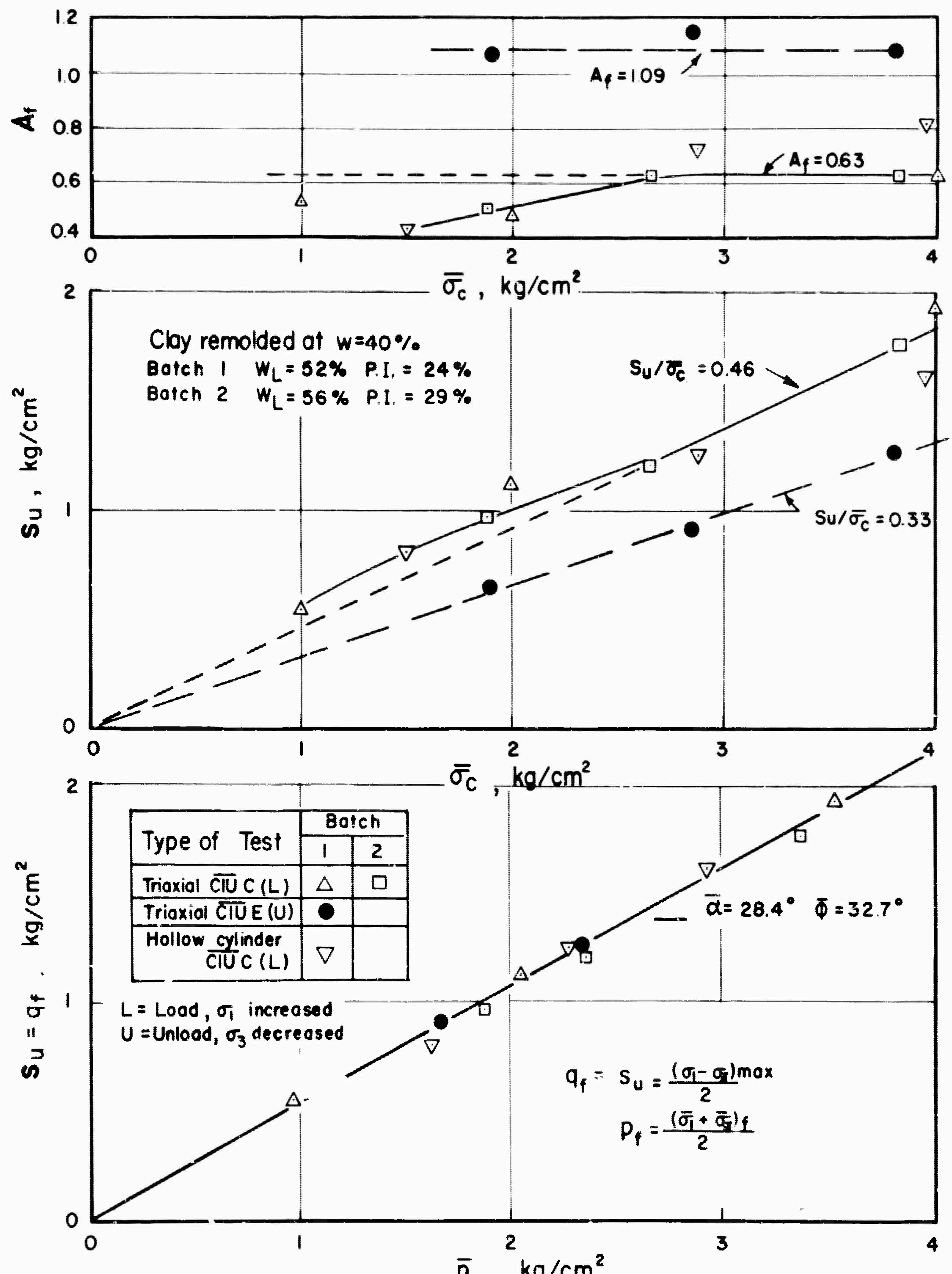
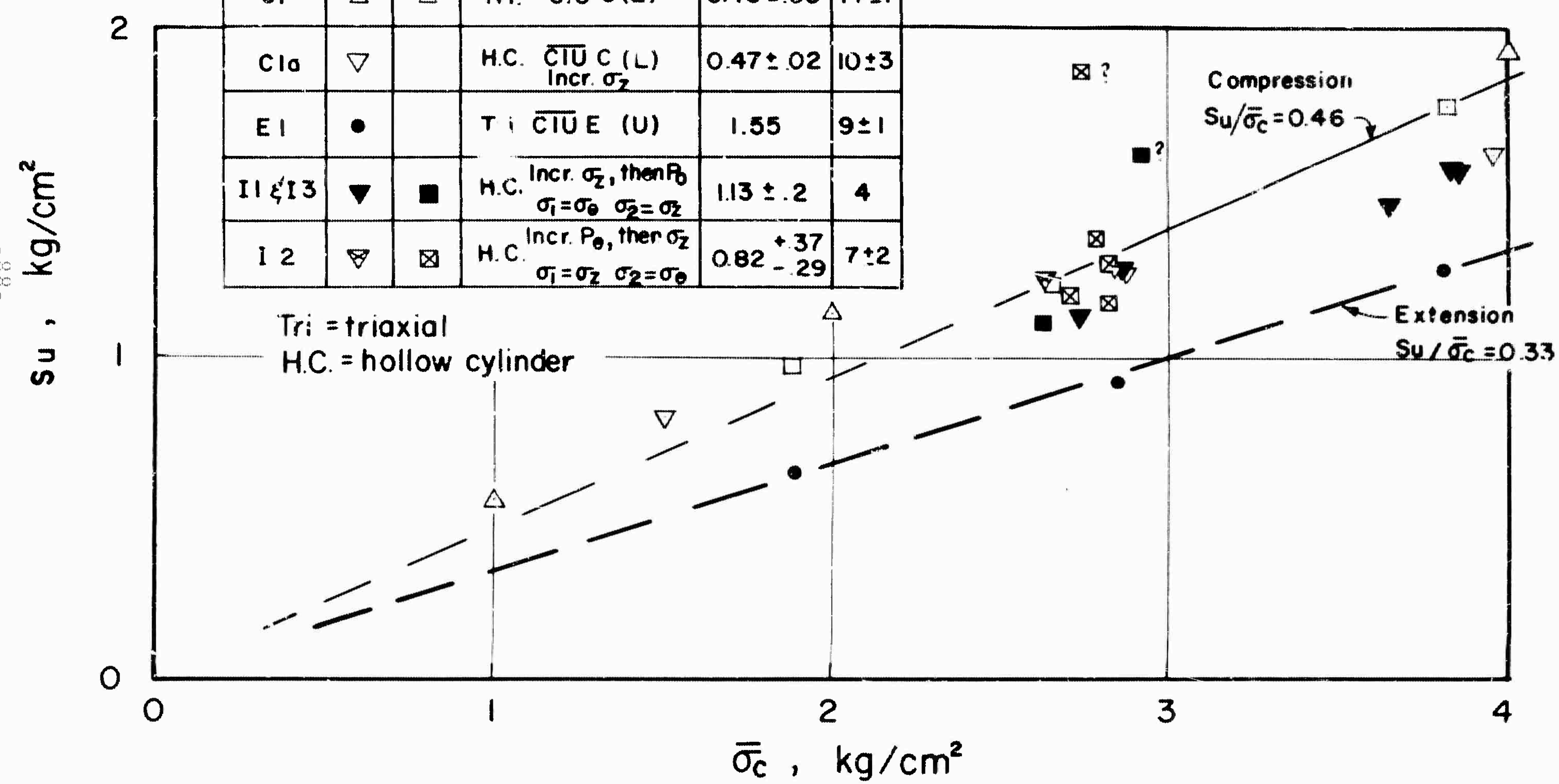


Fig. 2.18. $\overline{CIU}$ Triaxial and Hollow Cylinder Tests on Remolded Sault Ste Marie Clay (S_u vs $\bar{\sigma}_c$)

(Wu, Loh and Malvern, 1963)



Test	Batch 1	Batch 2	Test Description	$\bar{\sigma}_{2f}/\bar{p}_f$	ε_f %
C1	△	□	Tri. $\overline{CIU}$ C (L)	0.46 ± .03	14 ± 1
C1a	▽		H.C. $\overline{CIU}$ C (L) Incr. σ_z	0.47 ± .02	10 ± 3
E1	●		Tri $\overline{CIU}$ E (U)	1.55	9 ± 1
I1 & I3	▼	■	H.C. Incr. σ_z, then P_o; $\sigma_1 = \sigma_\theta$ $\sigma_2 = \sigma_z$	1.13 ± .2	4
I2	⊽	⊠	H.C. Incr. P_θ, then σ_z; $\sigma_1 = \sigma_z$ $\sigma_2 = \sigma_\theta$	$0.82^{+.37}_{-.29}$	7 ± 2

Fig. 2.19 $\overline{CIU}$ Triaxial and Hollow Cylinder Tests on Remolded Sault Ste Marie Clay (q_f vs $\bar{p}_f$)

(Wu, Loh & Malvern, 1963)

Test	Batch 1	Batch 2	Description	$\bar{\sigma}_{2f}/\bar{p}_f$	ϵ_f (%)
C1	△	□	Triaxial $\overline{CIU}$ C(L)	0.46 ± .03	14 ± 1
C1a	▽		Hollow cylinder $\overline{CIU}$ C (L)	0.47 ± .02	10 ± 3
E1	●		Triaxial $\overline{CIU}$ E (U)	1.55	9 ± 1
I1 & I3	▼	■	Hollow cylinder $\sigma_1 = \sigma_\theta$ $\sigma_2 = \sigma_z$	1.13 ± .2	4
I2	⊽	⊠	Hollow cylinder $\sigma_1 = \sigma_z$ $\sigma_2 = \sigma_\theta$	$0.82^{+.37}_{-.29}$	7 ± 2

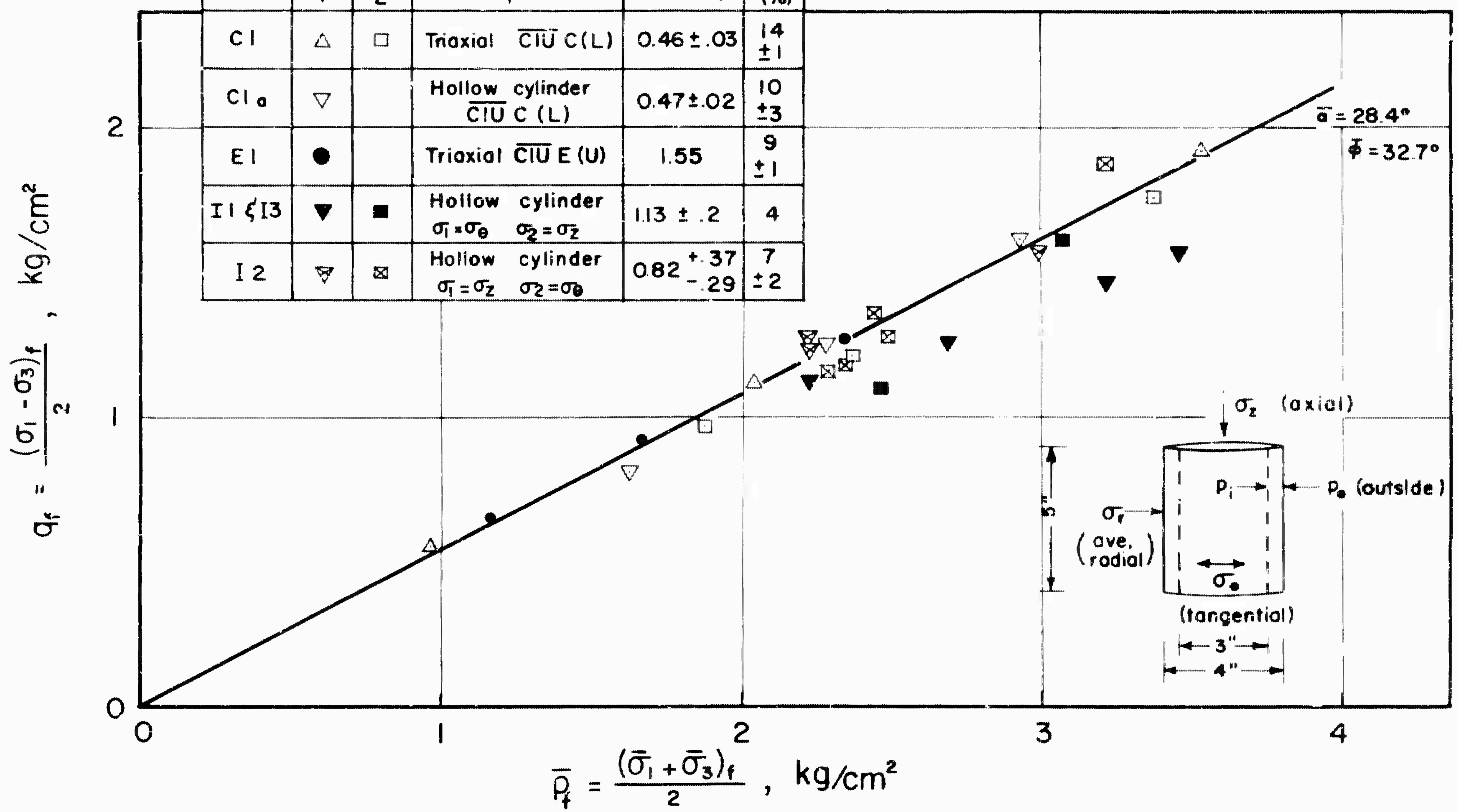


Fig. 2.20 Effect of Intermediate Principal Stress on Undrained Strength Behavior of Remolded Kaolinite (Broms and Cosbarian, 1964)

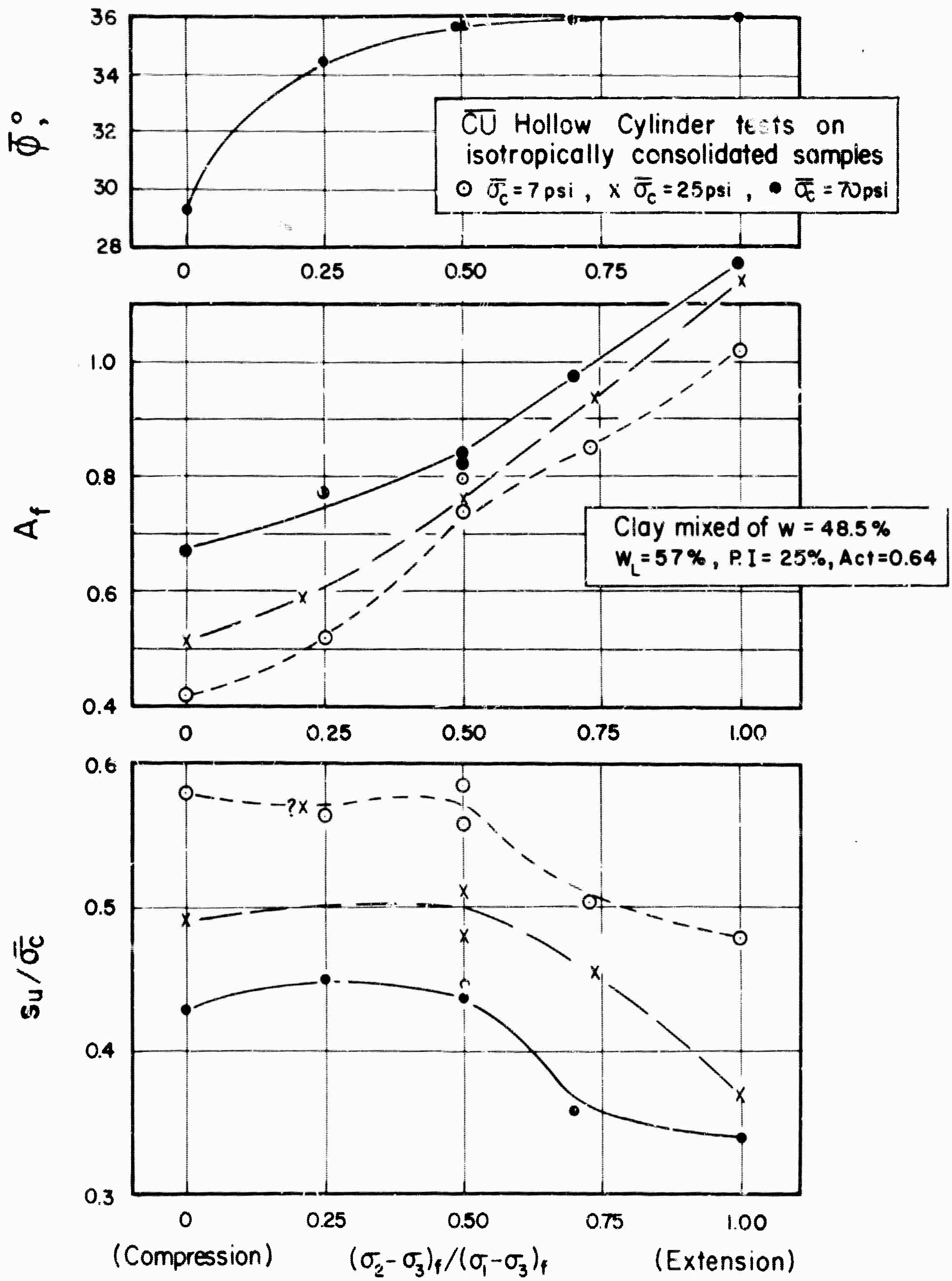


Fig. 2.21 Rotation of Principal Planes Along The Failure Surface for a Strip Footing on a Normally Consolidated Clay

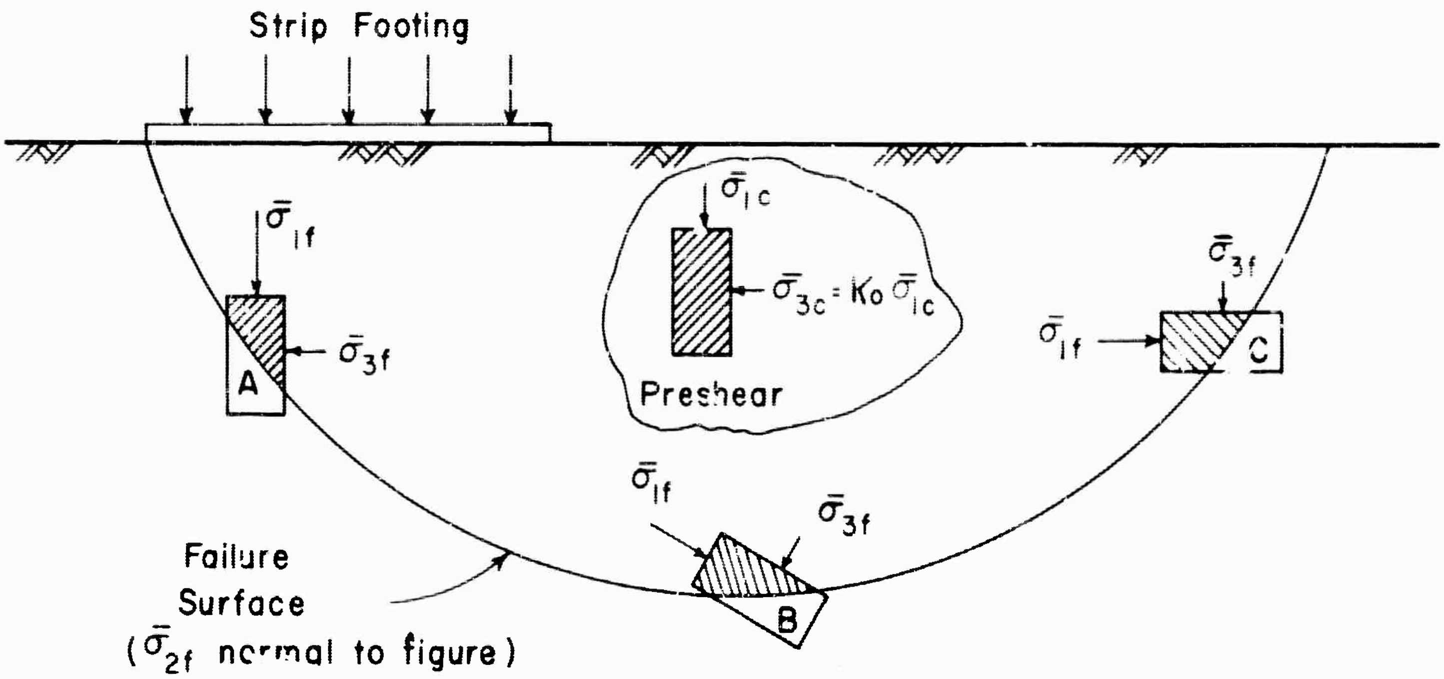


Fig. 2.22 Rotation of Principal Planes for an In Situ Vane Shear Test in a Normally Consolidated Clay

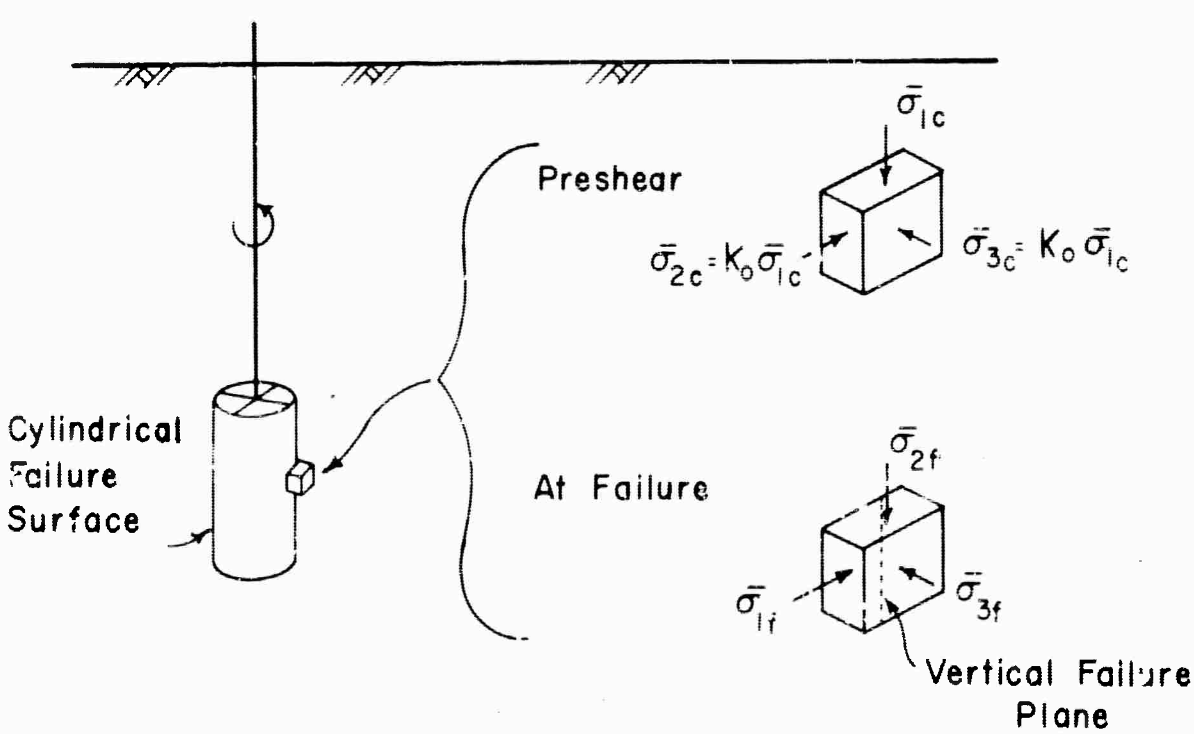
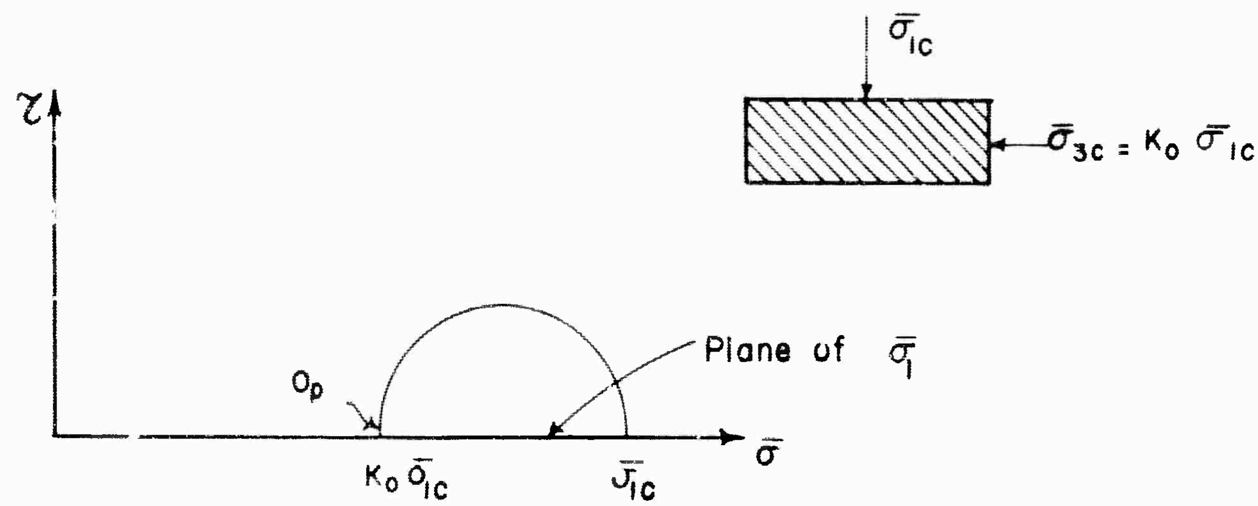


Fig. 2.23 Rotation of Principal Planes in a Direct Shear Test on a Normally Consolidated Clay

At Consolidation



At Failure

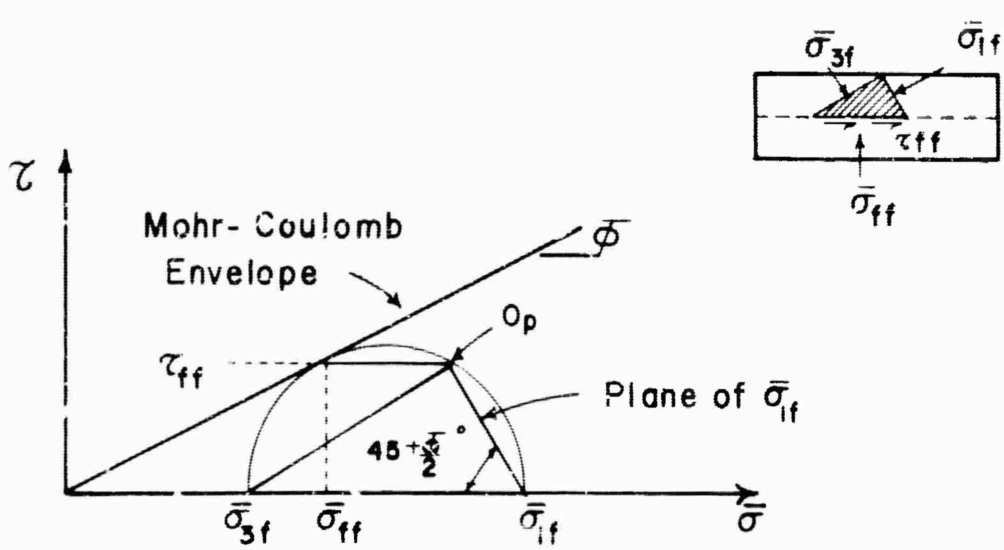


Fig. 2.24 Effect of Direction of Failure Plane on Undrained Strength Behavior of Remolded Kaolinite (Broms and Casbarian, 1964)

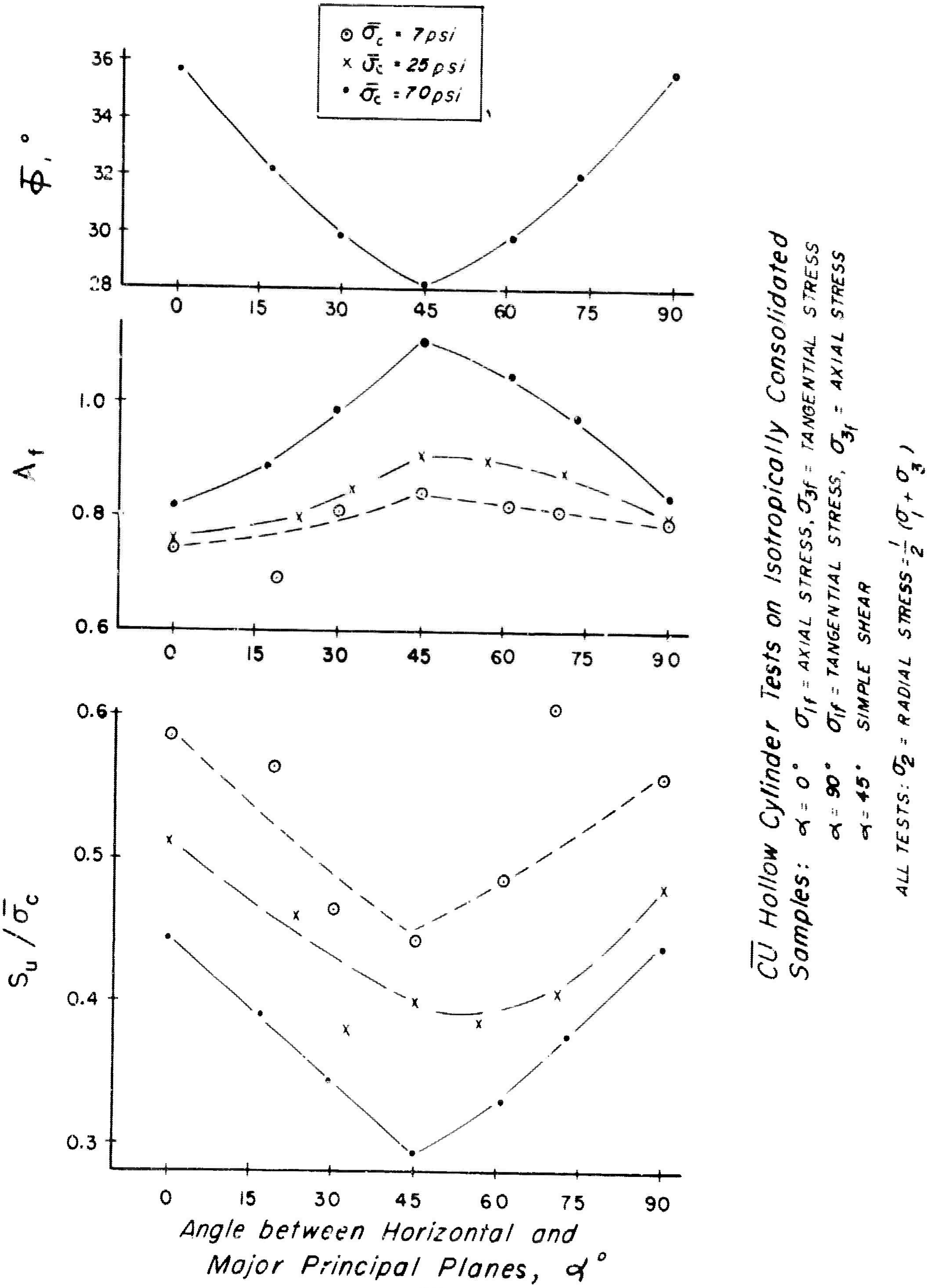


Fig. 2.25 Effect of Total Stress Path on Effective Stress Path for Undrained Triaxial Tests on N.C. Undisturbed Kawasaki Clay II

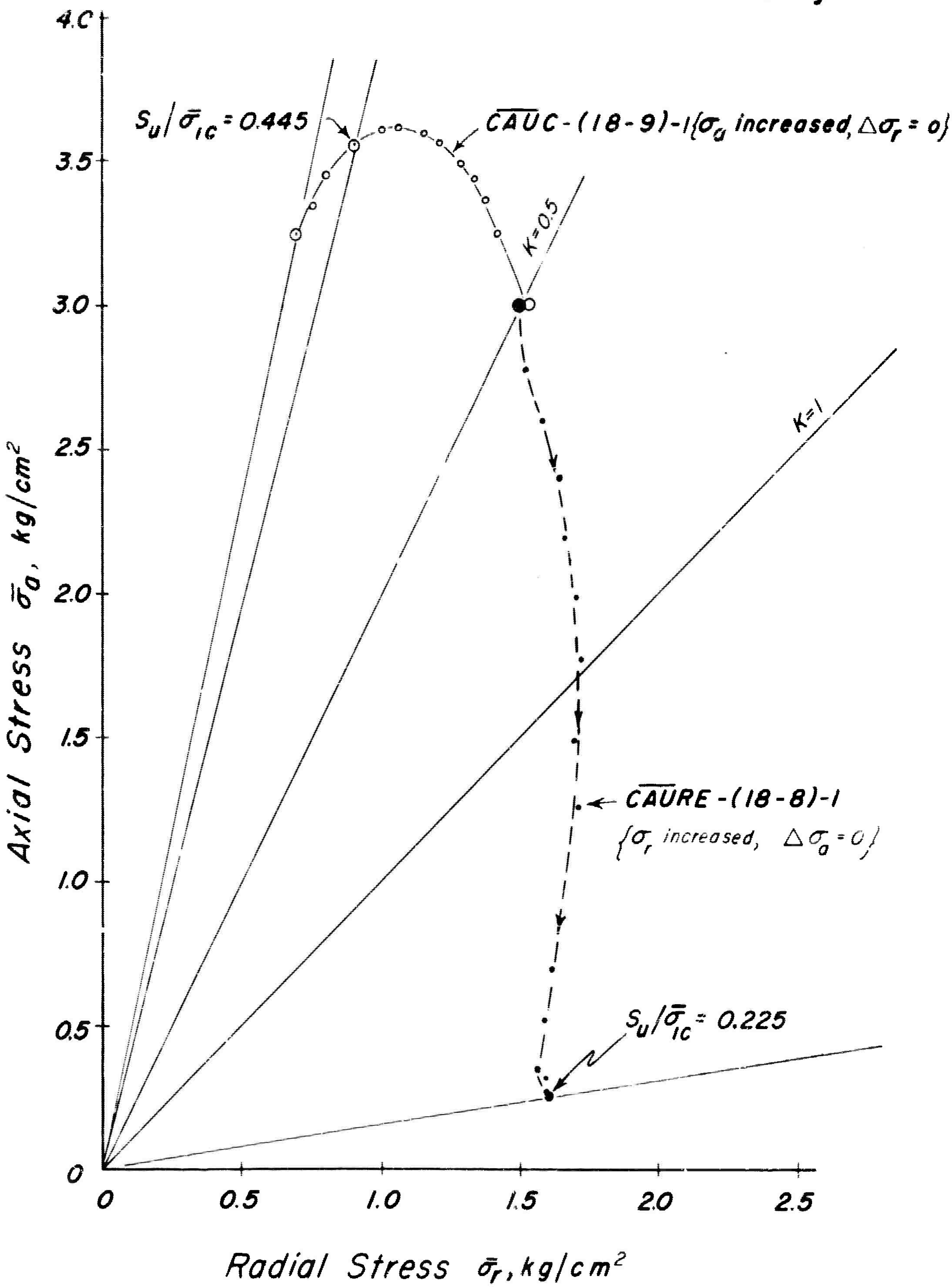


Fig. 2.26 Effect of Total Stress Path on Stress-Strain Behavior for Undrained Triaxial Tests on N.C. Undisturbed Kawasaki Clay II

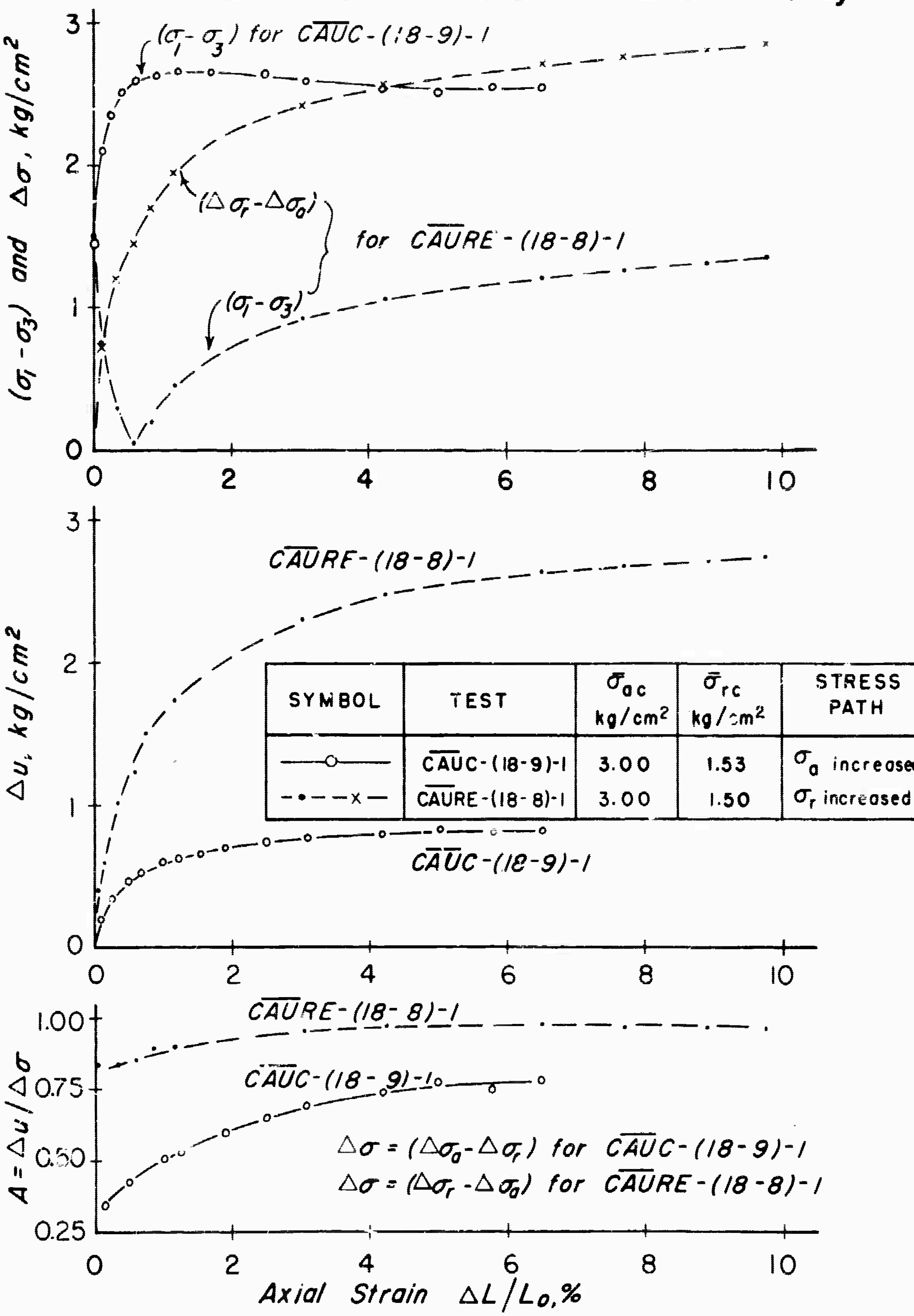
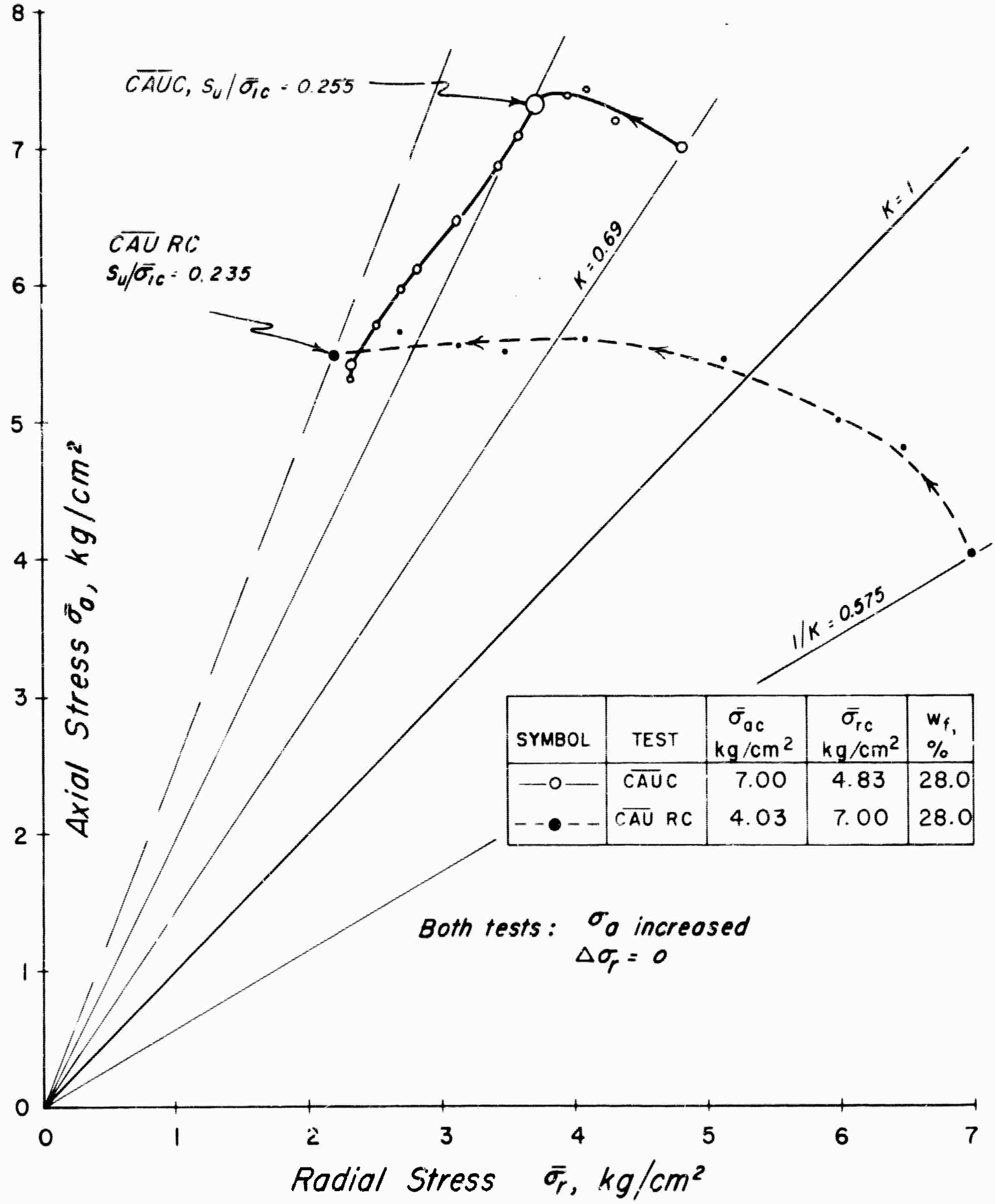


Fig. 2.27 Effect of Total Stress Path on Effective Stress Paths for Undrained Triaxial Tests on N.C. Remolded Vicksburg Buckshot Clay



SYMBOL	TEST	$\bar{\sigma}_{ac}$ kg/cm²	$\bar{\sigma}_{rc}$ kg/cm²	w_f, %
—o—	$\overline{CAUC}$	7.00	4.83	28.0
--●--	$\overline{CAU}$ RC	4.03	7.00	28.0

Fig. 2.28 Effect of Total Stress Path on Stress-Strain Behavior for Undrained Triaxial Tests on N.C. Remolded Vicksburg Buckshot Clay

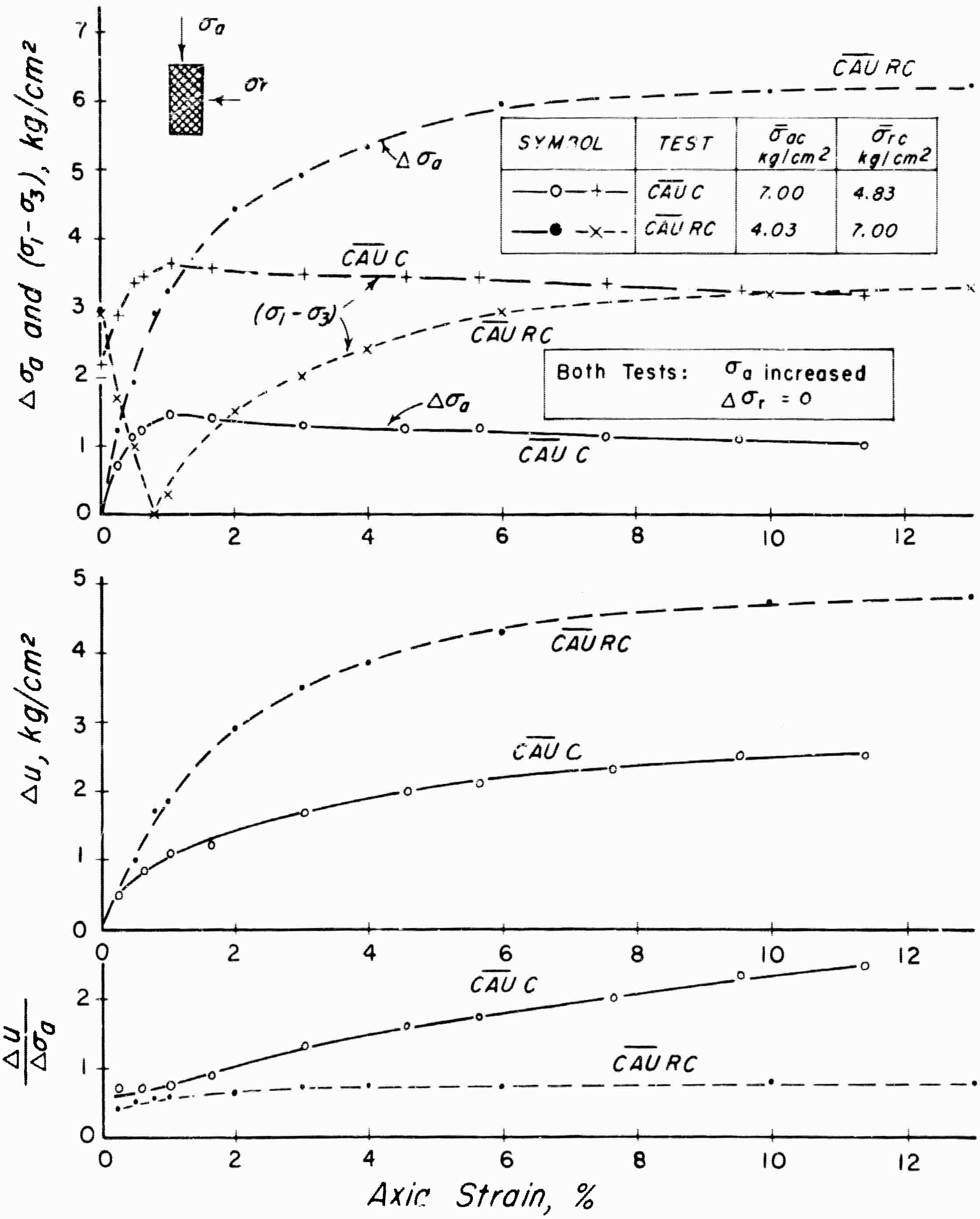


Fig. 2.29 Strength Parameters from $\overline{CU}$ Triaxial and Simple Shear Tests on N.C. Undisturbed Manglerud Clay (Landva, 1962)

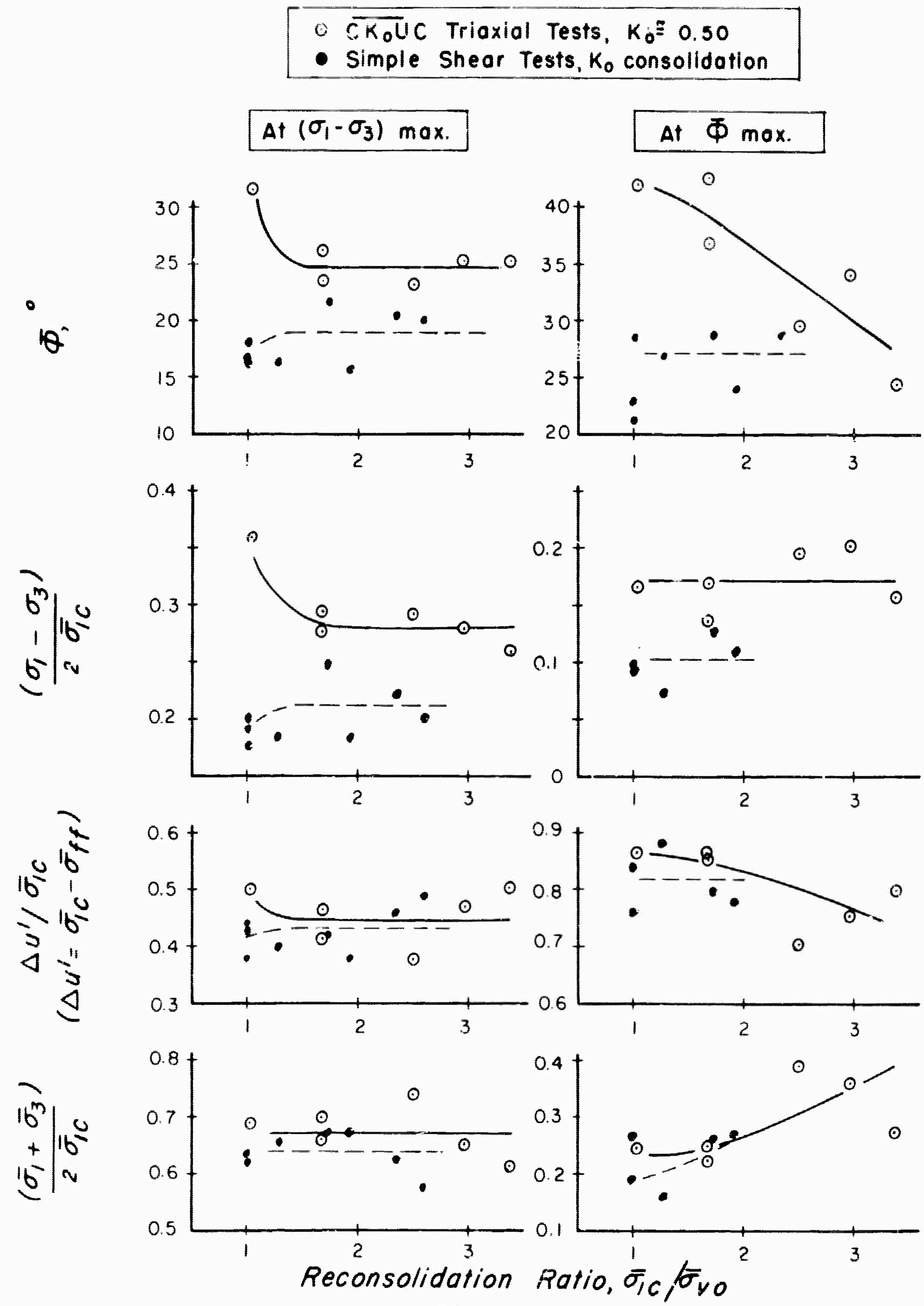


Fig. 2.30 Stress vs Strain from $\overline{CU}$ Triaxial and Simple Shear Tests on N.C. Undisturbed Manglerud Clay (Landva, 1962 F.175 Fig. 3)

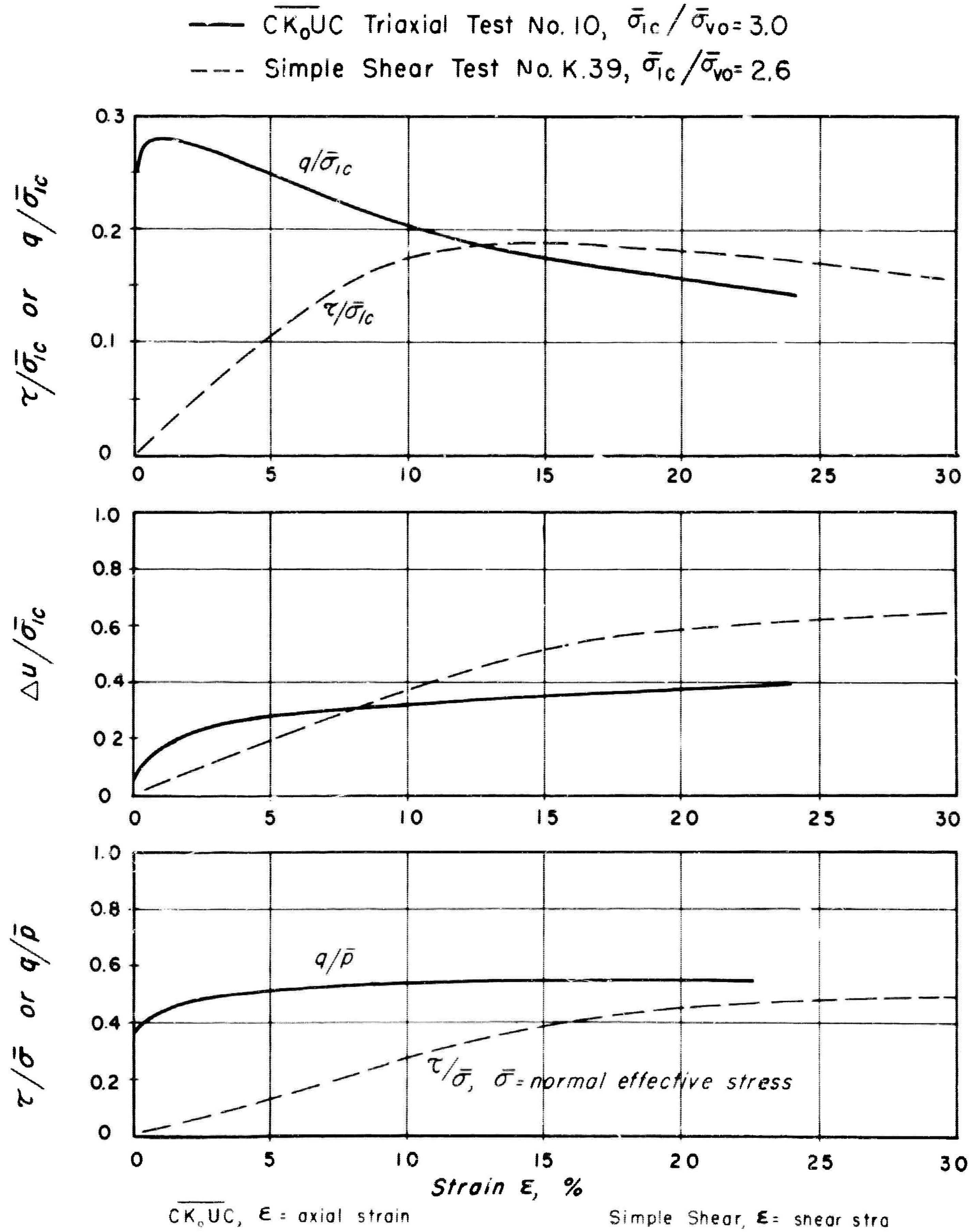
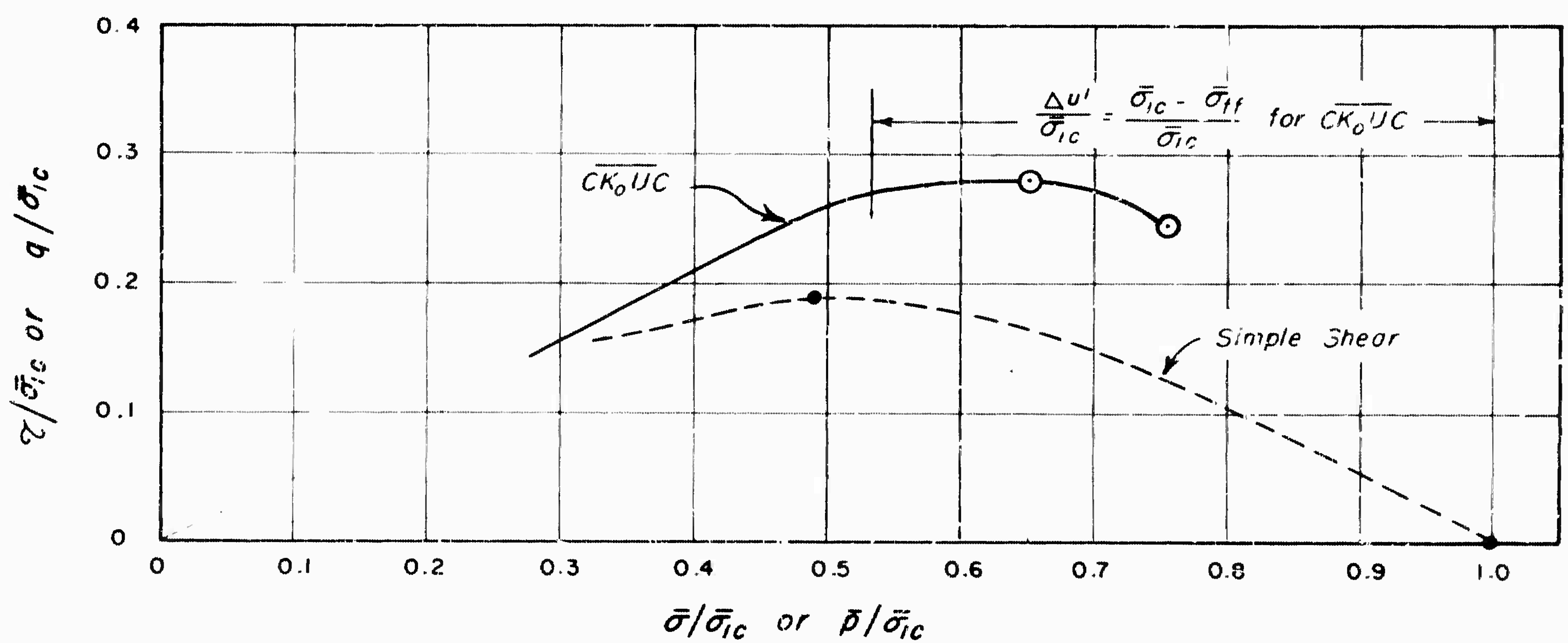


Fig. 2.31 Effective Stress Paths from $\overline{CU}$ Triaxial and Simple Shear Tests on N.C. Undisturbed Manglerud Clay (Landva, 1962)

$\overline{CK_oUC}$ Triaxal Test No.10, $\bar{\sigma}_{1c}/\bar{\sigma}_{vo} = 3.0$ Plot $q/\bar{\sigma}_{1c}$ vs $\bar{p}/\bar{\sigma}_{1c}$

Simple Shear Test No. K.39, $\bar{\sigma}_{1c}/\bar{\sigma}_{vo} = 2.6$ Plot $\tau/\bar{\sigma}_{1c}$ vs $\bar{\sigma}/\bar{\sigma}_{1c}$ where τ and $\bar{\sigma}$ are shear and normal stresses on horizontal plane



CHAPTER 3

TEST PROCEDURES FOR TRIAXIAL TESTS ON BOSTON BLUE CLAY

3.1 BOSTON BLUE CLAY

Boston blue clay is generally found in thick deposits (up to 70 feet or more) underlying the Greater Boston area. The upper portion of the deposit is heavily overconsolidated, probably due to desiccation, whereas the lower portion is usually normally consolidated. It is a highly variable silty clay with liquid limits usually ranging from 30 to 60% and plastic limits from 20 to 35%. The per cent clay size typically varies between 20 and 40%. Some portions contain appre-quantities of sand. The specific gravity of the solids is usually 2.80 ± 0.02.

The composition of Boston blue clay is predominately illite and quartz with some chlorite.

In order to obtain a uniform supply of clay, remolded rather than natural soil was used. The clay was prepared by consolidating a fairly dilute slurry of soil in a large container to a moderate pressure and then trimming triaxial specimens from the large sample. This procedure yielded a clay with strength properties similar in many respects to those of a natural normally consolidated clay of moderate sensitivity. Thus, this remolded clay acted differently than most remolded clays.

3.1.1 Sample Preparation

A large batch of soil 9.5 inches in diameter and 4.5 to 5 inches high was prepared in the laboratory following the steps described below:

1. 10,000 g. of dry powder of clay (the original clay was obtained from field pits and subsequently

air dried and ground in the laboratory) was mixed into a slurry with tap water at a water content of approximately 400%.

2. The slurry was thoroughly mixed and passed through No. 100 and No. 200 mesh sieves.

3. The salt content of the mixing water was increased to 16 - 17 g./liter by adding NaCl; after the clay-water system flocculated, excess salt water was siphoned off and a dilute slurry obtained with a water content of approximately 140%. The salt (NaCl) content of the pore fluid and the dilute slurry was determined prior to one-dimensional consolidation.

4. The slurry was heated to 70°C, with continuous stirring, and then placed under vacuum in an evacuated consolidometer; the apparatus and its operation is described in detail by Wissa (1961, p. 87). Dow Corning Silicone lubricant was applied liberally on the inside of the cylinder of the consolidometer to reduce friction during consolidation. The deaired slurry was mixed carefully in the consolidometer before lowering the piston in place.

5. The slurry was one-dimensionally consolidated from a water content of approximately 140% to $\bar{\sigma}_{1c} = 1.50$ kg./cm^2. When consolidation was completed, the batch was extruded as described by Wissa (1961, pp. 87-88).

6. Upon extrusion, the batch was cut with a wire saw into three chunks and stored in Mobile BB Transformer oil in a humid room.

7. Individual samples were cut and trimmed for each test from this supply.

3.1.2 Uniformity of Batches

Average water content is plotted in Fig. 3.1 as a function of height for the two clay batches which were the main sources of supply used in this investigation. In addition, one sample was taken from a similar batch of Boston blue clay designated S-4. Water content measurements from the trimmings of this sample are also plotted in the same figure.

3.1.3 Classification Data

Classification data (performed according to Lambe, 1951) from batches Nos. S-5 and S-6 are given in Table 3-1. Available data are also given for batch S-4. The grain-size distribution curve for the clay used for batch No. S-5 is plotted in Fig. 3-2.

3.2 TEST PROGRAM

A series of consolidated-undrained triaxial tests were run with pore pressure measurement on samples cut from previously consolidated batches of Boston blue clay. Prior to "sampling" in the laboratory, the clay was subjected to a consolidation pressure of 1.5 kg./cm^2. The major principal consolidation pressure, $\bar{\sigma}_{1c}$, used in the triaxial tests was never below 4.00 kg./cm^2. Therefore, all the samples should behave as if they were normally consolidated.

The program is described in Table 3-2. In reiteration, the variables investigated for their effects upon behavior in undrained shear were:

1. Value of K
2. Value of σ_2 at failure
3. Effect of perfect sampling
4. Value of major principal consolidation pressure

5. Controlled stress versus controlled strain during shear.

3.3 TRIAXIAL TEST PROCEDURES

3.3.1 Setup of Samples

All compression tests were performed in Clockhouse triaxial cells. Tests involving rotation of principal planes and axial extension were all run in Geonor, S.A. [manufactured in Oslo, Norway, and described by Andresen and Simons (1960)] triaxial cells. Only bottom end drainage was used throughout the investigation. All samples were of 1.41 inch diameter.

The drainage lines leading from the bottom pedestal of the base of the triaxial cell were flushed through with deaired water and a burette attached to one of the lines to record volume changes during consolidation. The other drain line was shut. A red rubber sleeve was forced on the pedestal, always assuring that it could be rolled up upon placement of the porous stone and the sample to just cover the entire thickness of the stone, but not cutting into the clay. Following placement on the pedestal of a saturated porous stone, the sample was mounted, and the filter paper drains [eight vertical strips in all compression tests and three spiral drains in tests involving reorientation and extension, consisting of Whatman's No. 54 paper, described by Bishop and Henkel (1962)], were put in place such that their ends reach down to the body of the pedestal over the side of the porous stone. The red rubber sleeve was then rolled up.

The rubber membranes (Trojan prophylactics) were rolled over the top loading cap and secured with 2 or 3 O-rings. The top cap was placed on top of the sample, which was already on the pedestal with filter paper drains. One of the membranes was carefully rolled down over the sample and red rubber sleeve, which now prevented direct contact between the membrane and the porous stone. Dow Corning Silicone Lubricant was lightly applied to the outside of the membrane and the red rubber sleeve. The second membrane was then rolled down and both were fastened securely to the bottom pedestal with 2 or 3 O-rings.

Placement of the plexiglass cylinder and the loading piston was arranged in two different ways:

1. In all compression tests (excluding those which involve reorientation of principal planes, i.e., consolidated $1/K_o$ and subsequently sheared in compression) the plexiglass cylinder was fastened to the base of the cell. It was filled with deaired water leaving about 60 cc. of air on top. The top 60 cc. was then filled with Teresso 85 (produced by Esso) oil to reduce leakage of water around the piston and to minimize piston friction during shear. The loading piston was then lowered through the bushing on top of the cell into the top loading cap. The apparatus was ready for consolidation.

2. In all tests involving the application of an axial upward force, (i.e., tests failed in extension or consolidated with $1/K_o$ stresses) the loading piston was fastened to the top loading cap before the sample was mounted on the pedestal of the base. Therefore, to put the plexiglass cylinder in place, the top bushing of the Geonor apparatus was removed to prevent the sample from being loaded. The top bushing was carefully lowered over the loading piston once the cylinder was fastened to the base and the cell filled with deaired water. Teresso 85 oil was then forced into the top 50-60 cc. of the cell with a small pump displacing the same amount of water from the cell, which was let out through the cell line

in the base of the apparatus.

3.3.2 Consolidation

Three ratios of minor to major consolidation pressure were applied before shear. These were: $K = 1$, K_o and $1/K_o$. Each will be treated separately. Constant confining pressures applied to the triaxial cell water were obtained from an adjustable mercury column [described by Bishop and Henkel (1962)] or an N.G.I. constant pressure cell [see Andresen and Simons (1960)].

Isotropic Consolidation. The confining pressure was applied to the water in the triaxial cell, the drainage line leading to the burette was opened, and the sample allowed to consolidate. Burette readings were generally taken at 1/2, 1, 2, 4, 8, etc., minutes after the valve to the burette was opened. Burette readings versus log time were recorded during each pressure increment to ensure that primary consolidation was completed and to detect the presence of leaks from the cell into the sample. Each pressure increment usually doubled the preceding one, beginning with 1.00 kg./cm^2, until the desired preshear consolidation pressure was reached. The axial shortening of the sample was measured by recording a change in a vertical reading (usually between top of piston and top of plate of cell) during consolidation.

K_o Consolidation. A nominal ratio of K_o was selected based on past experience with the same clay. This ratio, $\bar{\sigma}_{rc}/\bar{\sigma}_{ac}$, was equal to 0.54. The minor principal consolidation stress, $\bar{\sigma}_{3c} = \bar{\sigma}_{rc}$, was applied to the triaxial cell water, and the stress difference, $\bar{\sigma}_{1c} - \bar{\sigma}_{3c}$, acted on the top of the loading piston via a dead weight hanger [see Fig. 52 in Bishop and Henkel (1962)]. After the application of the pressures the drainage line leading to the burette was opened. Axial shortening in the sample was recorded by an extensometer (0.0001 inch divisions) resting on the crossbar of the hanger. Volume and length changes were plotted against log time during consolidation at 1/2, 1, 2, 4, 8, etc. minutes. The first stress increment was $\bar{\sigma}_{3c}/\bar{\sigma}_{1c} = 0.60/1.12$ kg./cm^2, followed by typical values of 1.5, 2.17, 2.72, 3.27, 3.61, 4.00, 4.66, 5.32 and 6.00 kg./cm^2 for $\bar{\sigma}_{1c}$; attempts were made to keep $\bar{\sigma}_{3c}/\bar{\sigma}_{1c}$ as close to 0.54 as possible throughout consolidation (actual preshear values of K are given for each test in the next chapter). At the end of consolidation 0.10 kg./cm^2 was added to the stress difference in an attempt to correct for the constraint provided by the vertical paper drains. Such corrections were not applied if spiral paper drains were used in a test as, for example, in all K_o consolidated tests followed by rotation of principal planes in extension (i.e., $\overline{C(K_o)URE}$ tests).

<u>$1/K_o$ Consolidation</u>: A nominal ratio of horizontal to vertical consolidation pressure was taken as the reciprocal value of K_o; this ratio, $1/K_o$, was equal to 1.84. The major consolidation pressure, $\bar{\sigma}_{1c}$, was applied to the water in the triaxial cell, whereas the minor consolidation pressure, $\bar{\sigma}_{3c}$, acted axially. To produce such a stress system, an upward force was applied during consolidation to the loading piston, thereby decreasing the axial stresses below the value of the all-around cell water pressure. Any desired stress difference could be obtained with a modification of the N.G.I. anisotropic loading arrangement [described in Fig. 10 in Andresen and Simons (1960)] mounted on top of the triaxial cell. The arrangement is shown also in Fig. 3.3. The drainage line leading to the burette was opened after the application of the load increment. The increase in length of the sample in the axial direction was measured with an extensiometer (0.0001 inch divisions) resting on the top bar of the hanger. Volume and length changes were plotted against log time at the same time intervals as previously noted for K = 1 and K_o consolidation. As the first stress increment, $\bar{\sigma}_{3c}/\bar{\sigma}_{1c}$ = 0.33/0.60 kg./cm^2 was usually applied, followed by typical values of 0.80, 1.10, 1.50 kg./cm^2 for $\bar{\sigma}_{1c} = \bar{\sigma}_{rc}$.

Overlapping vertical paper drains were used in tests which were to be sheared in axial compression, $\overline{C(1/K_o)URC}$ tests, and spiral paper drains in those which were sheared in extension $\overline{C(1/K_o)UE}$ tests.

3.3.3 Pore Pressure Measurement

The N.G.I. null system [a U shaped tube with a 1.3 mm. bore diameter filled with mercury, shown in Fig. 6 of Andresen and Simons (1960)] was used to measure pore pressures in the sample. The null system was thoroughly deaired and checked for leaks. The volume measuring burette was then removed from the end of the drainage line and the pore pressure measuring device attached to it. To ensure saturation of the sample, the sample was backpressured (the pore water in the sample pressurized through the null system to an arbitrary amount, usually 3.00 kg./cm^2 in this investigation) and the cell water pressure increased by the same amount. A period of a few hours usually elapsed before the system came to equilibrium.

The pore pressure response (P.P.R.) was measured when equilibrium was reached in the mercury U tube, indicating that the entire system adjusted itself to the increase in pore and cell pressures. Note that in tests wherein $K \neq 1$ during consolidation, a change in σ_r would change σ_a unless an axial force were applied either through the proving ring (if and when the sample is ready for shear) or the anisotropic loading arrangement. This was taken into account in every P.P.R. measurement to maintain constant effective stresses on the samples. The P.P.R. is the ratio of the increase in pore pressure, Δu, to the increase in chamber pressure $\Delta\sigma_c$. This ratio is the same as Skempton's B parameter. It was measured by determining the rise in pore pressure when the chamber pressure was increased by an arbitrary amount of 1.00 kg./cm^2. A P.P.R. greater than 90% was considered adequate to ensure a sufficient degree of saturation in the sample. If the P.P.R. was lower than 90% the system was backpressured for another hour or so to establish equilibrium in the system. Except in one case, this procedure always yielded 90% P.P.R. or greater at a backpressure of 3.00 kg./cm^2.

3.3.4 Undrained Shear

The Norwegian Geonor and Wykeham-Farrance loading frames were used for tests of the controlled strain type, whereas loads in the controlled stress type were applied through the N.G.I. anisotropic loading arrangement and/or a dead weight hanger.

Controlled Strain. In these tests a nominal strain rate of 1% per t_{90} was applied (t_{90} designates the time required for 90% consolidation in the triaxial apparatus) to ensure adequate pore pressure equalization in the sample. Shearing was carried on until both maximum stress difference and maximum obliquity had been reached or exceeded. Time required to reach failure under the rates of strain as shown in Appendix B differed according to type of test. The following times were typical: $\overline{CIUC}$ tests, 10 hours; $\overline{CIUE}$ tests, 7 - 8 days; $\overline{C(K_o)UC}$ and $C(K_o)$-$\overline{UU}C$ tests, 10 - 11 hours; $\overline{C(K_o)}\overline{U}RE$ tests 7 - 8 days; $\overline{C(1/K_o)UE}$ tests, 5 days; $\overline{C(1/K_o)}\overline{U}RC$ tests, 5 days.

Pore pressure was measured either directly through the null indicator or indirectly. Direct measurement of pore pressure proved to be the most convenient method in all tests involving axial extension and/or reorientation of principal planes. In simple compression tests, however, the pore pressure during shear was maintained constant and equal to the back pressure while the cell pressure was varied to keep the level of the mercury unchanged in the U shaped tube of the null system. Hence, in these tests the reported values of pore pressure change are actually changes in cell pressure. This method of testing is common practice in the Soil Mechanics Laboratory at M.I.T. It has been called a "constant volume test."

Readings of elapsed time, change in pore or cell pressure, sample length, and proving ring were taken during shear. Length changes were measured with an extensiometer (0.0001 in./division). The axial force was measured with Wykeham-Farrance proving rings calibrated for both axial compression and extension (usually about 0.06 kg./0.0001 inch deflection).

In tests involving axial extension a special connection was provided between the proving ring and loading piston to withstand an axial pull. An extension test in progress is shown in Fig. 3.4.

The following types of tests were run with controlled strain during undrained shear: $\overline{\mathrm{CIUC}}$, $\overline{\mathrm{CIUE}}$, $\overline{\mathrm{C(K_o)UC}}$, $\overline{\mathrm{C(K_o)\text{-}UUC}}$, $\overline{\mathrm{C(1/K_o)UE}}$ and $\overline{\mathrm{C(1/K_o)URC}}$ and $\overline{\mathrm{C(K_o)URE}}$.

Controlled Stress. In the first three or four increments, ten per cent of the estimated load increase necessary to cause failure was applied. In subsequent increments this was reduced to five per cent or to an even lower value near failure. Readings of pore pressure and axial strain were repeated several times after the application of each load increment until equilibrium was approximately reached. The next loading was then applied to the sample. In both $\overline{\mathrm{CIUE}}$ and $\overline{\mathrm{C(K_o)URE}}$ tests this practice allowed 40 to 60 minutes for one increment. The stress controlled $\overline{\mathrm{C(1/K_o)URC}}$ tests were sheared at a much slower rate, i.e., increments were applied at every 6 - 12 hours, or sometimes even less frequently.

Pore pressures in those tests were always measured directly through the null system.

Stress controlled shear was used in the following types of tests: $\overline{\mathrm{CIUE}}$, $\overline{\mathrm{C(K_o)URE}}$ and $\overline{\mathrm{C(1/K_o)URC}}$.

3.3.5 Measurements and Calculations

Before the sample was set up in the triaxial apparatus, its weight and length were measured and recorded. Five water content determinations were made from trimmings, three from the sides, one from the top, and one from the bottom. The weighted average of these was taken as the initial water content.

After failure, all stresses were removed from the sample at constant volume and the cell was then dismantled. The final weight, length, and the circumference at the top, middle, and bottom of the sample were all measured before a final water content determination was made on the whole sample.

Length and volume changes were recorded during consolidation. Change in length of the sample during shear was measured to obtain continuous strain readings.

All the above data were analyzed in a trial and error program, and adjustments were made where necessary to ensure a consistent set of values for weight of solid particles and volume changes during consolidation as computed from initial and final measurements of the total weight and water content of samples, and burette readings during consolidation. These calculations were carried out for each test and the results are presented on a separate data sheet for each test in Appendix B.

To compute the stress difference at any stage during shear, the area of cross section of the sample must be known. The conventional area correction is based on the assumption that the sample deforms at constant volume as a right circular cylinder, and an expression is obtained of the form, $A = \frac{A_o}{1 \pm \epsilon}$, where A_o designates the initial preshear area, and $\epsilon = \frac{\Delta L}{L_o}$, where L_o is the initial length and ΔL is the change in length. The sign of ϵ changes depending on whether the sample is sheared in axial compression or extension. This method was applied in $\overline{\text{CIUC}}$, $\overline{\text{C(K}_o\text{)UC}}$ and $\overline{\text{C(K}_o\text{)-UUC}}$ tests throughout shear.

Preliminary calculations in tests involving axial extension or compression carried to larger strains suggested that the area of the average cross section of the sample as determined by the above expression is in error. The order of magnitude of the discrepancy becomes intolerable beyond an axial strain of 6 to 7%. Therefore, in all tests which were sheared in axial extension, or in compression following reorientation of principal planes, the stress difference beyond ϵ = 6 - 7% was computed according to area changes that were obtained from an assumed parabolic distribution of area of the sample. Photographs of extension test specimens are shown in Figs. 3.5 and 3.6.

Calculated values of stress difference as obtained from either of the above two methods were then corrected for the estimated effects of the filter paper drains and piston friction. The filter strip correction was applied only when vertical paper drains had been placed on the sample. These corrections, as functions of axial strain, are shown below.

Corrections to Stress Difference

Filter Paper Correction

% Strain	Correction to $(\sigma_1 - \sigma_3)$, kg./cm^2
0-2	$(\% \epsilon / 2\%) \times (0.10)$
> 2	0.10

Piston Friction Correction

% Strain	Correction: % $(\sigma_1 - \sigma_3)$
0-2	0
2-4	0.5
4-6	1.0
6-8	1.5
8-10	2.0
10-12	2.5
etc.	

These corrections are only treated as an approximation to actual values. They were based on experience in the Soil Mechanics Laboratory at M.I.T. and data published by Bishop and Henkel (1962).

TABLE 3.1

CLASSIFICATION DATA ON BOSTON BLUE CLAY

	Batch No.		
	S-4	S-5	S-6
Specific gravity, G_s	-	2.78	-
Average water content, %	-	31.2	30.3
Liquid limit, %	32.6	33.3	32.8
Plastic limit, %	19.5	20.4	20.3
Plasticity Index, %	13.1	12.9	12.5
Liquidity Index	-	0.81	0.80
Shrinkage limit (remolded), %	-	16.9	-
Salt content, g/ℓ NaCℓ	-	16.8	16.0

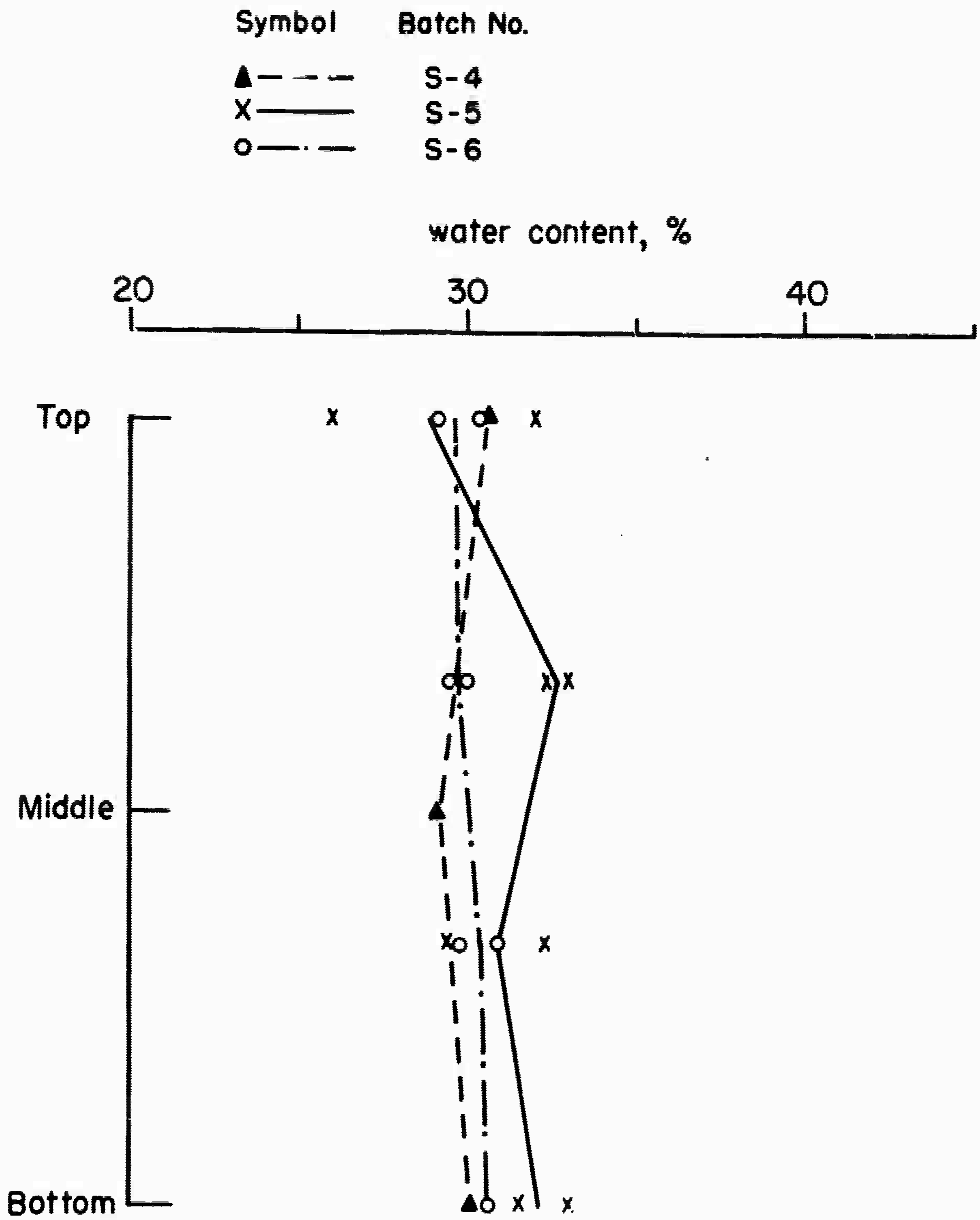
TABLE 3.2

TEST PROGRAM OF CONSOLIDATED-UNDRAINED TRIAXIAL TESTS ON BOSTON BLUE CLAY

Test No. and Type	$\bar{\sigma}_{ac}$ kg/cm^2	$\bar{\sigma}_{rc}$ kg/cm^2	K = $\bar{\sigma}_{rc}/\bar{\sigma}_{ac}$	Total Stress Path $\Delta\sigma_a$	Total Stress Path $\Delta\sigma_r$	Controlled σ	Controlled ϵ	Batch No.
$\overline{CI}\overline{U}C$-1	6.0	6.0	1.0	Incr.	Decr.		✓	S-5
$\overline{CI}\overline{U}C$-2	4.0	4.0	1.0	Incr.	Decr.		✓	S-5
$\overline{CI}\overline{U}C$-3	8.0	8.0	1.0	Incr.	Decr.		✓	S-5
$\overline{CI}\overline{U}E$-1	4.0	4.0	1.0	0	Incr.	✓		S-5
$\overline{CI}\overline{U}E$-2	4.0	4.0	1.0	Decr.	0		✓	S-6
$\overline{CI}\overline{U}E$-3	6.0	6.0	1.0	Decr.	0		✓	S-6
$\overline{C(K_o)}\overline{U}C$-1	4.0	2.16	.54	Incr.	Decr.		✓	S-5
$\overline{C(K_o)}\overline{U}C$-2	6.0	3.24	.54	Incr.	Decr.		✓	S-6
$C(K_o)$-$\overline{UU}$-1	4.0	2.16	.54	Incr.	Decr.		✓	S-5
$C(K_o)$-$\overline{UU}$-2	6.0	3.24	.54	Incr.	Decr.		✓	S-5
$\overline{C(K_o)}\overline{U}RE$-1	4.0	2.16	.54	0	Incr.	✓		S-4
$\overline{C(K_o)}\overline{U}RE$-2	4.0	2.16	.54	0	Incr.	✓		S-5
$\overline{C(K_o)}\overline{U}RE$-4	4.0	2.16	.54	Decr.	0		✓	S-6
$\overline{C(K_o)}\overline{U}RE$-5	6.0	3.24	.54	Decr.	0		✓	S-6
$\overline{C(1/K_o)}\overline{U}E$-1	2.16	4.0	1.85	Decr.	0		✓	S-6
$\overline{C(1/K_o)}\overline{U}E$-3	3.24	6.0	1.85	Decr.	0		✓	S-6
$\overline{C(1/K_o)}\overline{U}RC$-2	2.16	4.0	1.85	Incr.	0	✓		S-5
$\overline{C(1/K_o)}\overline{U}RC$-4	2.16	4.0	1.85	Incr.	0	✓		S-6
$\overline{C(1/K_o)}\overline{U}RC$-5	2.16	4.0	1.85	Incr.	0		✓	S-6
$\overline{C(1/K_o)}\overline{U}RC$-6	3.24	6.0	1.85	Incr.	0		✓	S-6

Note: Pore pressure measured in all tests.

Fig. 3.1 Variation in Water Content of Batches of Boston Blue Clay



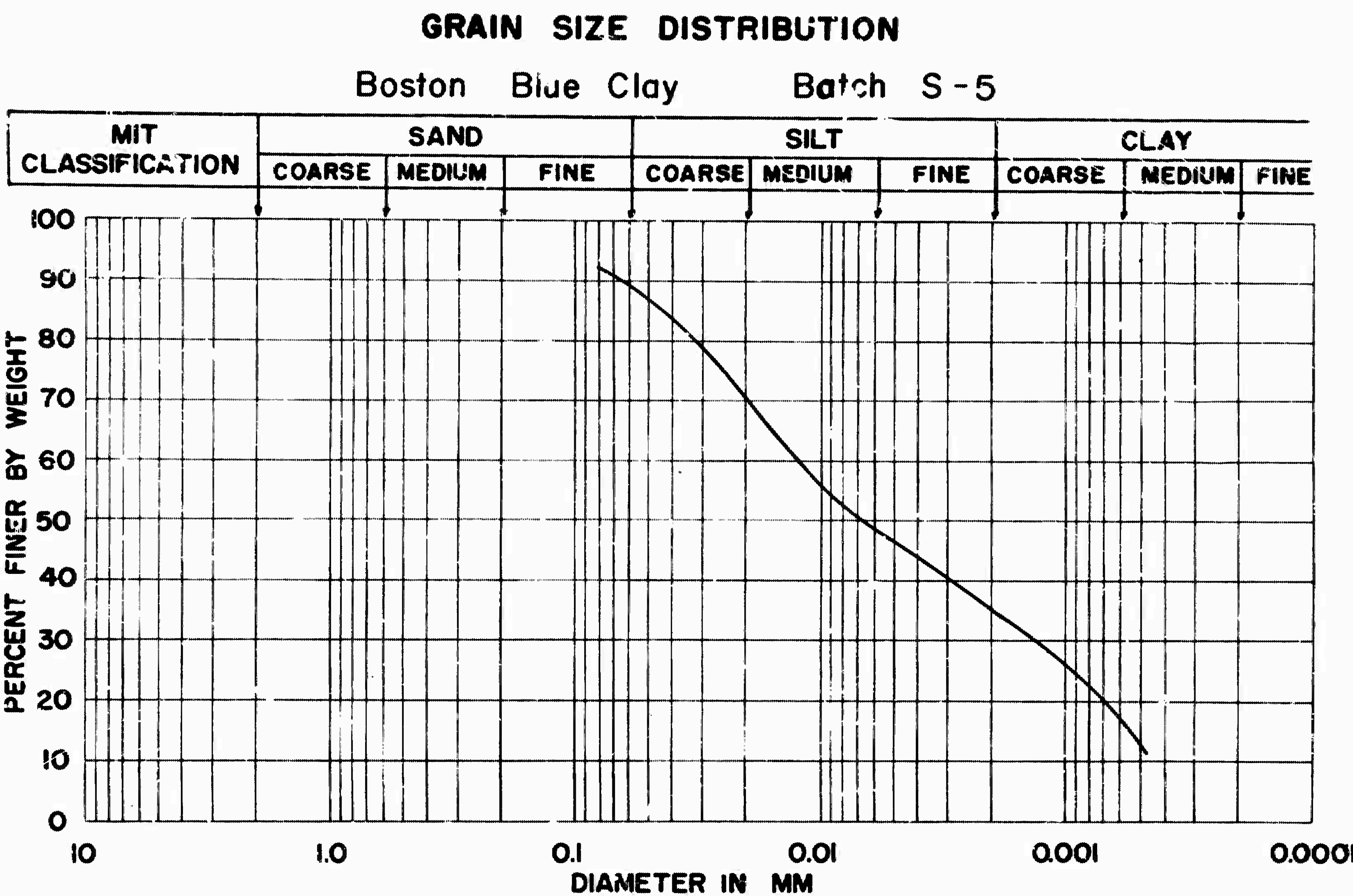

GRAIN SIZE DISTRIBUTION
Boston Blue Clay Batch S-5
MIT CLASSIFICATION
SAND
COARSE
MEDIUM
FINE
SILT
COARSE
MEDIUM
FINE
CLAY
COARSE
MEDIUM
FINE
PERCENT FINER BY WEIGHT
100
90
80
70
60
50
40
30
20
10
0
10
1.0
0.1
0.01
0.001
0.0001
DIAMETER IN MM

Fig. 3.2

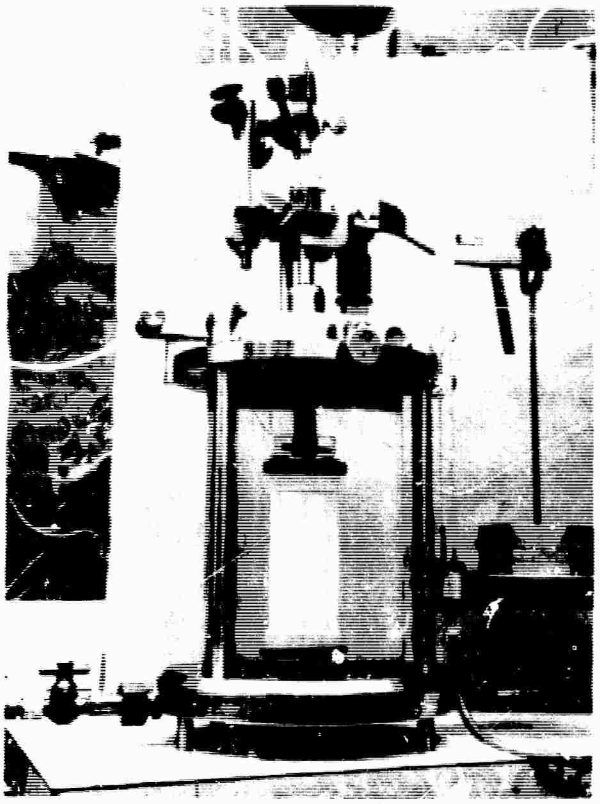


Fig. 3.3

$1/K_o$ Consolidation

In N.G.I. Triaxial

Cell

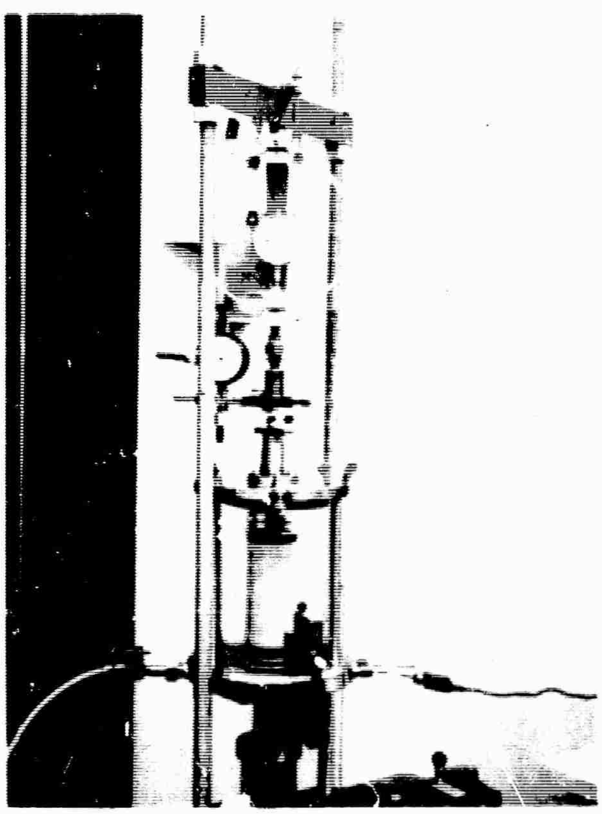


Fig. 3.4

Strain Controlled

Extension Test

In Progress

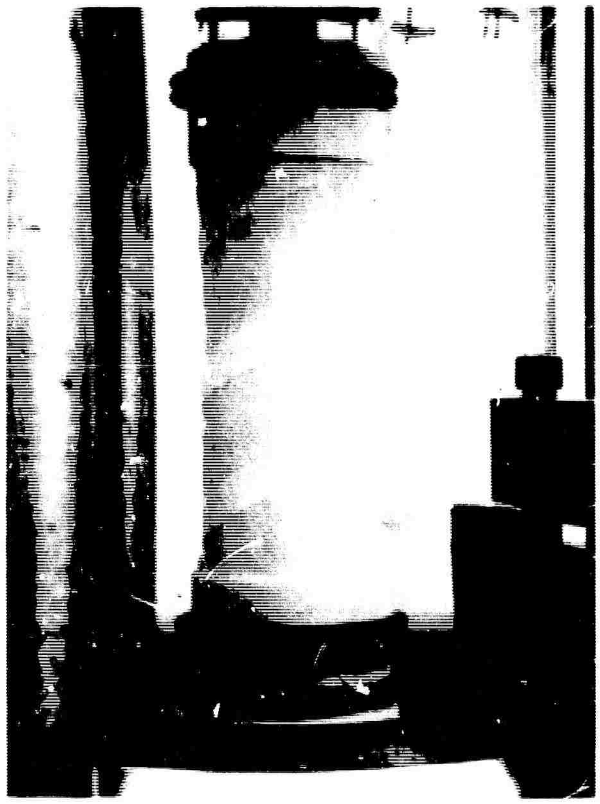


Fig. 3.5

$\overline{C(1/K_o)U}E$ Test

In Progress At

Approximately 5%

Axial Strain

Fig. 3.6

Oven-dried

Sample, Test

No. $\overline{CIU}E$-1

CHAPTER 4

PRESENTATION AND ANALYSIS OF TRIAXIAL TEST DATA ON NORMALLY CONSOLIDATED BOSTON BLUE CLAY

4.1 PRESENTATION OF RESULTS

The program of consolidated-undrained triaxial tests on normally consolidated samples of remolded Boston blue clay (BBC) was described in Section 3.2, Table 3.2, and Fig. 1.6. The results of the tests are summarized as follows:

Table 4.1 - Summary of stresses and strains at maximum stress difference and at maximum obliquity, and water content and consolidation stress data;

Table 4.2 - Information on water contents, consolidation stress increments, time of consolidation under the last increment, time to 1% strain and to strain at failure (maximum stress difference);

Figures 4.1 to 4.4 - Effective stress paths (plot of $\bar{\sigma}_a$ versus $\bar{\sigma}_r$) from strain controlled tests with isotropic, K_o, and $1/K_o$ consolidation and from tests with stress control during shear;

Figures 4.5 to 4.7 - Summary of strength data (q_f versus $\bar{p}_f$ and $\bar{\sigma}_{1c}$ versus s_u and A_f) for tests with isotropic, K_o, and $1/K_o$ consolidation respectively.

Appendix B presents tables of properties during consolidation and tabulated stress-strain data for each of the twenty tests. Stress-strain plots for each test are given in Appendix C. Water content and volumetric strain data are plotted versus major principal consolidation stress for $K = 1$, K_o, and $1/K_o$ consolidation in Appendix D.

4.2 EFFECT OF WATER CONTENT AT FAILURE

The plots of water content versus consolidation stress in Appendix D show considerable scatter due to variations in initial water content and compressibility (the volumetric strain plots also show considerable scatter) and because of the problem of accurately determining water contents.* What is the effect of water content variations on the undrained strength behavior of samples consolidated to the same pressure? Data are presented in Fig. 4.8 which suggest that water content variations are relatively unimportant. The figure plots A_f, $s_u/\bar{\sigma}_{1c}$ and $(\bar{\sigma}_1/\bar{\sigma}_3)$ versus water content at failure from $\overline{\text{CIUC}}$ and $\overline{\text{CAUC}}$ tests with the same major principal consolidation stress run on a variety of batches (some with fresh water and some with salt water as the pore fluid) over a three year period. Even though the spread in water contents corresponds to changes in consolidation stress on the order of one kg./cm^2, there is no consistant variation in the parameters with variation in water content at failure for a given pore fluid and type of test. (It is interesting to note that the pore fluid salt concentration also had a relative minor effect on strength behavior even though the water contents were altered appreciably.)

It is concluded that scatter in water contents at failure probably does not have a significant effect on the undrained strength behavior of this clay-water system.

However, in comparing data from various tests it is important to compare tests having approximately the same time of consolidation under the last increment. In fact, it is better to

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* Water contents from trimmings often varied; changes in water content during consolidation are difficult to determine because of inaccurate volume change readings and possible variations in the degree of saturation; the samples undoubtedly imbibed some water during disassembly (Henkel and Sowa, 1963).

compare averages from several tests using normalized parameters because variations between tests of the same type (due to slight differences in test procedures) are often as large as differences between different types of tests.

4.3 EFFECT OF CONSOLIDATION STRESS

Many normally consolidated clays exhibit normalized behavior (p. 10-11 of Part I) in that the strength behavior in terms of dimensional parameters [$(\sigma_1 - \sigma_3/\bar{\sigma}_{1c}$, $(\bar{\sigma}_1/\bar{\sigma}_3)$, $\Delta u/\bar{\sigma}_{1c}$ and A versus axial strain and $\bar{\sigma}_a/\bar{\sigma}_{1c}$ versus $\bar{\sigma}_r/\bar{\sigma}_{1c}$] is independent of the magnitude of the consolidation stress. The data on BBC for strain controlled tests with $\bar{\sigma}_{1c}$ = 4 and 6 kg./cm^2 do not show a consistant trend. In general, for samples failed in compression, an increased consolidation stress has little effect on $s_u/\bar{\sigma}_{1c}$ and A_f (the $\overline{\text{CIUC}}$ tests show considerable scatter), although maximum obliquity often increases. On the other hand, for samples failed in extension, an increased consolidation stress causes a 11 - 14% decrease in $s_u/\bar{\sigma}_{1c}$, and an increase in A_f; the data at maximum obliquity are probably unreliable because of necking.

It is concluded that consolidation stress does not have a large influence on strength behavior. In comparing strength parameters from the different types of tests, average values from the tests with $\bar{\sigma}_{1c}$ = 4 and 6 kg./cm^2 will generally be employed. These values are presented in Table 4.3.

4.4 CONTROLLED STRAIN VERSUS CONTROLLED STRESS TESTS

Both strain controlled and stress controlled tests were run for three types of tests. The stress-strain curves are compared in Figs. 4.9 through 4.11 for the $\overline{\text{CIUE}}$, $\overline{\text{C(K}_o\text{)URE}}$, and $\overline{\text{C(1/K}_o\text{)URC}}$ tests respectively. Table 4.1 summarized the data at maximum stress difference and at maximum obliquity. In all cases, the stress controlled tests yielded a lower $s_u/\bar{\sigma}_{1c}$ ratio (by 11 - 25%) and a higher value of A_f; stress difference versus strain was decreased, at least after several tenths per cent strain, but excess pore pressures versus strain were little altered.

The basic reason for the relatively large differences in undrained strength is not clear because of differences in the consolidation times and strain rates. For example, comparing the tests failed in extension, the stress controlled tests had a shorter time of consolidation under the last increment (3 days versus 6 to 12 days) and a smaller total volumetric strain during consolidation. It is known (Bailey, 1961; Ladd, 1961; Bjerrum and Lo, 1963; Ladd, 1964) that aging will increase the stress-strain modulus and undrained shear strength, while having little effect on excess pore pressures. On the other hand, the stress controlled tests were strained more rapidly and reached $(\sigma_1 - \sigma_3)_{max.}$ more quickly (10 - 13 hour versus 23-120 hour) than the strain controlled tests. This difference would tend to cause higher undrained strengths in the stress controlled tests (Bjerrum, et al, 1958; Crawford, 1959; Casagrande and Wilson, 1951; Richardson and Whitman, 1963).

Comparing the tests failed in compression, the stress controlled tests had about the same time of consolidation and volumetric strain, but a much slower average strain rate (110 versus 12 hours to reach 1% strain; 240 versus 76 hours to reach failure), which may have been an important factor contributing to the lower strength.

In summary, the stress controlled tests yielded lower values of s_u and stress-strain modulus (at larger strains), but practically identical excess pore pressures. The reason for this behavior is not clear. It may in part be caused by differences in aging or in the time to reach failure. Or possibly, the rapid strain accompanying each increment of loading in the stress controlled tests causes an additional rupture of interparticle bonds and "creep" that does not show up during relatively uniform straining.

4.5 EFFECT OF INTERMEDIATE PRINCIPAL STRESS

The effect of the intermediate principal stress, σ_2, is shown by a comparison of tests failed in compression ($\sigma_2 = \sigma_3 = \sigma_r$) with those failed in extension ($\sigma_2 = \sigma_1 = \sigma_r$). Data are available for isotropic consolidation ($\overline{\text{CIUC}}$ versus $\overline{\text{CIUE}}$ tests) and for anisotropic consolidation ($\overline{\text{C(K}_0\text{)UC}}$ versus $\overline{\text{C(1/K}_0\text{)UE}}$ tests).

For isotropic consolidation, the data are summarized in Table 4.3 and Fig. 4.5. The effective stress paths in Fig. 4.12, and the stress-strain curves in Fig. 4.13 show that shear in extension (relative to shear in compression):

1. Decreased s_u by 14%;
2. This decrease is caused by a large increase in excess pore pressures (best seen in Figs. 4.12 or 4.13);
3. $\Delta u/\bar{\sigma}_c$ at a given strain was generally 0.13 ± 0.03 higher; the A parameter at a given stress difference was higher by more than one-third (the theoretical value for a linear elastic material, also see p. 54 of Part I), except near failure;
4. The friction angle at $(\sigma_1 - \sigma_3)_{max.}$, $\bar{\Phi}_u$, may have increased somewhat (there were considerable scatter in the extension test data);
5. The stress-strain modulus, $E = (\sigma_1 - \sigma_3)/\epsilon_{axial}$, was increased by approximately 50% at low stress levels; the modulus in terms of strain in the direction of the major principal stress would be even higher because radial strains are only one-half of the axial strain (since $\Delta V = 0$, $\epsilon_a + 2\epsilon_r = 0$);
6. At maximum obliquity, $\bar{\Phi}$ was increased by 6°; however, the values of $\bar{\Phi}$ from the extension tests are suspect because necking at large strains may have produced significant errors in the calculated values of axial stress.

Comparisons of extension and compression tests on anisotropically consolidated samples are shown in Table 4.3 and Figs. 4.14 and 4.15. The trends are very similar to those observed with the $\overline{CIU}$ tests, in that shear in extension:

1. Decreased s_u, but only by 6 ± 6%;
2. The excess pore pressures were significantly increased, except at large strains;
3. $\overline{\phi}_u$ was increased slightly;
4. The stress-strain modulus probably increased (the strains were too small to accurately assess the difference);
5. $\overline{\phi}$ at maximum obliquity increased slightly (since maximum obliquity in extension occurred at relatively low strains and hence prior to necking, the values of $\overline{\phi}$ should be reliable).

Whereas the $\overline{CIU}$ compression and extension tests should have the same water content at failure, since both were isotropically consolidated under identical pressures, such is not the case with the $\overline{CAU}$ compression and extension tests. The compression tests were consolidated with K_o stresses ($\overline{\sigma}_{rc} = 0.53\ \overline{\sigma}_{ac}$) while the extension tests were consolidated with $1/K_o$ stresses (i.e., $\overline{\sigma}_{rc} = 1.9\ \overline{\sigma}_{ac}$). The average effective stress in the latter tests was therefore greater than that in the K_o tests. The resultant differences in volumetric strain are shown in Fig. 4.16. For a given value of major principal consolidation stress, the volumetric strain increased in going from $K = K_o$ to $K = 1$ to $K = 1/K_o$. The lower water contents in the $\overline{C(1/K_o)UE}$ tests may explain in part why the extension tests on anisotropically consolidated samples did not produce as large effects as observed with the isotropically consolidated samples.

In summary, an increase in the intermediate principal stress from triaxial compression to triaxial extension:

1. Decreases the undrained shear strength because of large increases in excess pore pressure;

2. Increases the stress-strain modulus at low stress levels;

3. Increases the friction angle at both $(\sigma_1 - \sigma_3)_{max}$. and $(\bar{\sigma}_1 - \bar{\sigma}_3)_{max.}$, although the exact difference is difficult to measure;

4. Increases the volumetric strain during consolidation.

All of the above effects produced significant differences, at least for research studies.

4.6 EFFECT OF ANISOTROPIC CONSOLIDATION

The effect of anisotropic consolidation on stress versus strain is shown in Figs. 4.17 and 4.18 from undrained compression and extension tests respectively. Effective stress paths in terms of axial and radial stresses are presented in Figs. 4.19 and 4.20. These data, and the summary of test results in Table 4.3, show that anisotropic consolidation (using average values):

1. Increased $s_u/\bar{\sigma}_{1c}$ by 16 to 21%;

2. Decreased $\bar{\phi}_u$ by 3 to 6°;

3. Decreased A_f by 0.35 to 0.67;

4. Decreased ϵ_f from 3.4 - 6% to only 0.2 - 3%.

At maximum obliquity, anisotropically consolidated compression tests produced the same friction angle, but a significantly lower shear stress. The opposite trends occurred with the extension tests; however, the problem of necking may be partially responsible for this behavior.

Reference to Fig. 4.16 shows that anisotropic consolidation had opposite effects on volume changes for compression and extension. Relative to K = 1, K_o stresses caused a smaller volume decrease and $1/K_o$ stresses caused a larger volume decrease. This difference in volume change behavior might partially explain why anisotropic consolidation in extension produced larger increases in strength than occurred in the compression tests. The larger excess pore pressures in the $\overline{\text{CIUE}}$ tests relative to the $\overline{\text{CIUC}}$ tests also explains why anisotropic consolidation yielded a greater strength increase for the extension tests.

The effect of "perfect" sampling from a K_o condition was covered in Section 2.3.

4.7 EFFECT OF ROTATION OF PRINCIPAL PLANES

4.7.1 Introduction

Due to limitations of the triaxial apparatus, it is not possible to study the effect of rotation of principal planes at a constant value of the intermediate principal stress σ_2. One can only investigate the influence of switching from compression to extension or vice versa. Consequently, σ_2 also changes, thus introducing another variable. However, by determining the effects of rotating from compression to extension and of rotating from extension to compression, one might ascertain upper and lower bounds of the effect of rotation at constant σ_2. An average of the two types of tests might also approximate the effect of rotation of principal planes in plane strain, where σ_2 is probably about midway between triaxial compression and extension.

Data are presented on two types of tests, $\overline{\text{C(K}_o\text{)URE}}$ and $\overline{\text{C(1/K}_o\text{)URC}}$, which will be referred to as RE and RC tests respectively for simplicity (RE = rotated and failed in extension; RC = rotated and failed in compression). In the RE test, the sample is K_o consolidated with $\bar{\sigma}_{ac} = \bar{\sigma}_{1c}$ and $\bar{\sigma}_{rc} = \bar{\sigma}_{2c} = \bar{\sigma}_{3c}$. It is then sheared undrained by decreasing σ_a and/or increasing σ_r (whether σ_a is decreased or σ_r is increased has no effect on effective stresses or on stress versus strain for a given value of $(\Delta\sigma_r - \Delta\sigma_a)$; the total stress path only influences Δu, just as $\Delta\sigma_3$

only influences Δu in triaxial compression tests*). At failure, $\sigma_r = \sigma_1 = \sigma_2$ and $\sigma_a = \sigma_3$, i.e., a triaxial extension state of stress exists. This test simulates failure under the center line of a circular excavation [see Fig. 1.4(b)], and will be compared to failure under the center line of a circular footing [see Fig. 1.4(a)] as represented by a $\overline{C}(K_o)\overline{UC}$ test.

In the RC test, the sample is consolidated under $1/K_o$ stresses with $\bar{\sigma}_{rc} = \bar{\sigma}_{1c} = \bar{\sigma}_{2c}$ and $\bar{\sigma}_{ac} = \bar{\sigma}_{3c}$. It is sheared by increasing σ_a and/or decreasing σ_r so that the stress system at failure is triaxial compression with $\sigma_a = \sigma_1$ and $\sigma_r = \sigma_2 = \sigma_3$. This test does not simulate a field situation for a normally consolidated clay. However, such a stress path could occur under the center line of a footing on a highly overconsolidated clay (K_o would be greater than unity and hence σ_3 would act in the vertical direction prior to shear).

4.7.2 Test Results (only strain controlled tests are considered unless otherwise stated)

Effective stress paths in terms of axial and radial stresses are plotted in Figs. 4.2 and 4.3; typical stress-strain curves are compared in Figs. 4.21 and 4.22. Table 4.3 and Figs. 4.6 and 4.7 summarize conditions at failure.

For tests starting from K_o stresses, failure in extension, relative to failure in compression:

1. Decreased $s_u/\bar{\sigma}_{1c}$ by 53 ± 5% (the stress controlled tests produced strength which were 60% lower);
2. Increased the friction angle at maximum stress difference, $\bar{\phi}_u$, by 10° (again there is a problem

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* See pages 21-22 of Part I. This is a very important fact (as long as the B parameter equals one) which is often not comprehended.

due to necking) and the strain at failure by 30 fold;

3. Doubled the value of A_f (A is defined as $(\Delta u - \Delta\sigma_a)/(\Delta\sigma_r - \Delta\sigma_a)$ in the RE test because $\Delta\sigma_a$ is the change in the minor principal stress in terms of the stress increments applied during undrained shear);

4. Increased $\bar{\Phi}$ by several degrees and decreased $q/\bar{\sigma}_{1c}$ by 35% at maximum obliquity.

It is emphasized that these large changes are the result of differences in applied stresses during undrained shear; the consolidation stresses and hence the water contents at failure were essentially identical (within experimental accuracy). This means that the undrained strength of a normally consolidated sample of BBC below the center line of a circular excavation is less than one-half of the strength of an identical sample beneath the center line of a circular footing.

For tests starting from $1/K_o$ stresses, failure in compression, relative to failure in extension:

1. Increased $s_u/\bar{\sigma}_{1c}$ by 10% (the stress controlled tests produced strengths which were 10 to 15% lower);

2. Increased $\bar{\Phi}_u$ by 10^o (no problem with necking in this case) and the strain at failure by some 65 fold;

3. Cut the value of A_f in half;

4. Increased $\bar{\Phi}$ only slightly but more than doubled $q/\bar{\sigma}_{1c}$ at maximum obliquity.

Consequently, this test series produced results which were opposite in many important respects from those of the RE tests.

Possible reasons for this behavior will be presented in Section 4.7.3.

The type of stress system applied during shear also has a pronounced effect on stress-strain moduli*, as illustrated in Fig. 4.23. The applied stress, in terms of change in stress difference, is plotted against axial strain. The results of a UU test on a "perfect sample" are added for comparison. In tests without rotation, $\overline{C(K_o)UC}$ and $\overline{C(1/K_o)UE}$, a very small increment of applied stress produces failure, whereas the other samples can sustain a large increment of applied stress before failing.

Values of stress-strain modulus ($E = \Delta\sigma/\epsilon$, where ϵ = axial strain) divided by the major principal consolidation stress are tabulated below.

No.	Type of Test	$\Delta\sigma$**	$E/\bar{\sigma}_{1c}$ At $\epsilon = 0.1\%$	$E/\bar{\sigma}_{1c}$ At $\epsilon = 1\%$
1	$\overline{C(K_o)UC}$-1	σ_a increased	150	14
2	$\overline{C(K_o)URE}$-4	σ_r increased	270	64
3	$C(K_o)$ - $\overline{UUC}$-1	σ_a increased (from K = 1)	270	54
4	$\overline{C(1/K_o)UE}$-1	σ_r increased	160	12
5	$\overline{C(1/K_o)URC}$-5	σ_a increased	390	81
6	$\overline{CIUC}$-2, 3	σ_a increased	240	56 ± 4
7	$\overline{CIUE}$-1, 2, 3	σ_r increased	330 ± 60	48 ± 5

The values of modulus quoted above were based on axial strains. In test numbers 1, 3, 5 and 6, the axial strain coincides with the strain in the direction of the applied major principal stress, i.e., $E_1 = \Delta\sigma_1/\epsilon_1$. However, in test numbers 2, 4 and 7, the applied major

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* See Ladd (1964) for a more detailed treatment of stress-strain modulus.

** $\Delta\sigma$ for "loading" case; the same result would be obtained if the other stress had been decreased ("unloading" case).

principal stress acts in the radial direction. For these tests, the strain in the direction of $\Delta\sigma_1$ would be one-half of the axial strain since $\epsilon_a + 2\epsilon_r = 0$. Values of modulus in terms of major principal stress and strain would, therefore, be twice as large as the tabulated numbers.

4.7.3 Discussion

The RE tests (failure in extension starting from K_o stresses) produced a very large decrease in undrained strength whereas the RC tests (failure in compression starting from $1/K_o$ stresses) caused a relatively small increase in undrained strength. These opposite trends will be analyzed by comparing these tests with the results of a $\overline{UU}$ test on a "perfect sample." The reasoning is as follows.

The $\overline{C(K_o)URE}$ test (RE test) can be broken into two portions:

1. Perfect sampling from a K_o condition to achieve the isotropic stress $\bar{\sigma}_{ps}$;
2. An $\overline{UU}$ triaxial extension test starting from $\bar{\sigma}_{ps}$.

Likewise the $\overline{C(1/K_o)URC}$ test (RC test) can be divided into:

1. Perfect sampling from a $1/K_o$ condition to achieve the isotropic stress $\bar{\sigma}_{ps}$;
2. An $\overline{UU}$ triaxial compression test starting from $\bar{\sigma}_{ps}$.

Effective stress paths in terms of q and $\bar{p}$ are plotted in Fig. 4.24 for use in the comparisons.

Analysis of RE Test

It is obvious from Fig. 4.24(a) that the basic cause of the much reduced strength of the RE sample is the low effective stress at failure. Relative to the $\overline{C(K_o)\text{-}UUC}$ test (an $\overline{UU}$ triaxial compression test starting from $\bar{\sigma}_{ps}$ = 2.52 kg./cm^2), the $\overline{C(K_o)URE}$

test developed much higher pore pressures during shear.* This is to be expected because the RE test is sheared from $\bar{\sigma}_{ps}$ in triaxial extension. Data in Section 4.5 have already shown that an increase in σ_2 produces an increase in excess pore pressures. Stress paths in Fig. 4.12 from $\overline{\text{CIU}}$C and $\overline{\text{CIU}}$E tests illustrate this fact.

Let us compare the effect of σ_2 on the tests in Fig. 4.24(a) with that exhibited in Fig. 4.12.

Test	$q/\bar{\sigma}_{ps}$ or $q/\bar{\sigma}_c$ At $(\sigma_1 - \sigma_3)_{max.}$	$q/\bar{\sigma}_{ps}$ or $q/\bar{\sigma}_c$ At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max.}$
$\overline{\text{C(K}_o\text{)}}$URE-4	0.66/2.41 = 0.274	0.65/2.41 = 0.270
C(K$_o$)-$\overline{\text{UU}}$C-1	1.12/2.52 = 0.445	0.785/2.52 = 0.312
	Ratio = 0.62	Ratio = 0.87
$\overline{\text{CIU}}$E (Table 4.3)	0.245	0.240
$\overline{\text{CIU}}$C (Table 4.3)	0.285	0.275
	Ratio = 0.86	Ratio = 0.87

At large strains, i.e., at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max.}$, the effect of σ_2 on stress difference as observed in the $\overline{\text{CIU}}$ tests appears to explain the behavior of the RE test relative to the $\overline{\text{UU}}$ compression test. But at $(\sigma_1 - \sigma_3)_{max.}$, the RE test has a much lower strength than can be

* Note: Both of these tests should have shown identical stress paths between $K = K_o$ and $K = 1$ and hence both should have had identical values of $\bar{\sigma}_{ps}$. In the case of test $\overline{\text{C(K}_o\text{)}}$URE-4, only one data point exists between K_o stresses and the point of $q = 0.14$, $\bar{p} = 2.20$; an approximate path was sketched in. Fig. 4.2 also shows lack of agreement between the stress paths of $\overline{\text{C(K}_o\text{)}}$URE-5 and C(K$_o$)-$\overline{\text{UU}}$C-2. This disagreement probably reflects slight changes in testing rates, times of consolidation under the last increment, water contents, etc. In other words, differences in effective stress paths between K_o and $\bar{\sigma}_{ps}$ are due to experimental scatter.

accounted for simply on the basis of the effects of σ_2. In other words, the RE test undergoes a much larger decrease in effective stress during the early portions of shear in extension than would be expected from the results of the $C(K_o)$-$\overline{\overline{U}U}C$ test adjusted for the effects of σ_2 by using $\overline{CIUC}$ and $\overline{CIUE}$ tests.

Factors which might help to explain this large increase in pore pressure are listed below:

1. The rate of strain in the RE tests was much lower than that in the $C(K_o)$-$\overline{UU}C$ tests (t_f = 120 hr. versus only 1 - 2 hr.). Lower strain rates often cause increased excess pore pressures;

2. The water content of the RE tests (K_o consolidation) was higher than that of the $\overline{CIU}$ tests (see Fig. 4.16). Possibly σ_2 has a greater effect on samples with a higher water content;

3. The structure (clay fabric plus interparticle force system) of the sample in the RE test was first developed to resist a major principal stress acting in the axial direction. The structure must then change to resist a major principal stress acting in the radial direction during shear in extension from K = 1. Increased pore pressures are, perhaps, induced as the clay particles reorient themselves into a new fabric to better resist the new direction of stresses. In other words, rotation of principal planes causes a reorientation of clay particles and additional sliding among particles that contributes to increased pore pressures (i.e., decreased ability to carry effective stress at particle contacts).

<u>Analysis of RC Test</u>

Figure 4.24(b) shows the stress path from a $\overline{C(1/K_o)URC}$ test and adds, for comparison, a $\overline{UU}$ triaxial compression test

starting from $\bar{\sigma}_{ps} = 2.52$ [from Fig. 4.24(a)]. In this case, both the RC test and the $\overline{UU}$ test have the same stress system during shear, but the values of $\bar{\sigma}_{ps}$ are different because one sample started from a K_o condition and the other started from a $1/K_o$ condition. The strengths of the two tests are compared below.

Test	$q/\bar{\sigma}_{ps}$ At $(\sigma_1 - \sigma_3)_{max.}$	At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max.}$
$\overline{C(1/K_o)U}$RC-5	1.325/2.80 = 0.474	1.265/2.80 = 0.452
$C(K_o)$-$\overline{UU}$C-1	1.12/2.52 = 0.445	0.785/2.52 = 0.312
	Ratio = 1.06	Ratio = 1.45

Although the tests have approximately the same value of $s_u/\bar{\sigma}_{ps}$, the RC test attained this strength with a lower effective stress and a higher friction angle, $\bar{\phi}_u$. Looking at the stress paths, in Fig. 4.24(b), one notes that the RC test developed larger pore pressures at low strains, but smaller pore pressures at high strains relative to the $C(K_o)$-$\overline{UU}$C test. A possible explanation of this behavior is given below:

1. At low strains, the particles in the RC test are reorienting themselves to adjust to the new stress system (σ_1 acted in the radial direction between $K = 1/K_o$ and $K = 1$, and now acts in the axial direction) with a resultant increase in pore pressure, just as was hypothesized for the RE test;

2. At high strains, the sample acts as if it were slightly overconsolidated. In fact, it is "overconsolidated" with respect to a compressive stress system because the water content of the RC sample is considerably lower than that of the $C(K_o)$-$\overline{UU}$C sample. Figure 4.16 shows that $1/K_o$ consolidation leads to a greater decrease in volume than K_o consolidation. Hence the lower

relative water content of the RC sample could explain the higher effective stresses at higher strains and the increased friction angle.

It should be noted that the strain rate in the RC tests was again much lower than in the $C(K_o)$-$\overline{UU}C$ tests and yet the strengths were higher. This may mean that strain rates are not a major factor.

In summary, the effect of rotation of principal planes depends on the direction of the change. Failure in extension starting from K_o stresses produces a very large decrease in undrained strength that can be encountered in practice. Failure in compression starting from $1/K_o$ stresses produces an increase in undrained strength. These opposite trends can be explained in terms of differences in σ_2 at failure and in water contents at consolidation. However, rotation of principal planes per se does appear to cause an additional increase in excess pore pressure for normally consolidated Boston blue clay.

Note: Table continued

Table 4.1 Summary of $\overline{CU}$ Triaxial Tests on N.C. Boston Blue Clay

Stresses in kg/cm^2

Test No.	Initial w (%)	Final w (%)	Consolidation Stresses (1)	Controlled	At $(\sigma_1-\sigma_3)$max.						At $(\bar{\sigma}_1/\bar{\sigma}_3)$max.						Remarks
					ϵ (%)	s_u	$\bar{p}$	$\bar{\sigma}_1/\bar{\sigma}_3$	A_f (2)	$s_u/\bar{\sigma}_{1c}$	ϵ (%)	q	$\bar{p}$	$\bar{\sigma}_1/\bar{\sigma}_3$	A (2)	$q/\bar{\sigma}_{1c}$	
$\overline{CIU}C$-2	31.4	25.4	$\bar{\sigma}_c = 4.00$ $K = 1.00$	ϵ	2.8	1.23	2.58	2.82	1.08	.308	10.7	1.15^5	2.08^5	3.48	1.33	.289	
$\overline{CIU}C$-1	30.4	23.4^5	$\bar{\sigma}_c = 6.00$ $K = 1.00$	ϵ	3.5	1.57^5	3.24^5	2.88	1.37	.263	10.1	1.50	2.73	3.44	1.59	.250	
$\overline{CIU}C$-3	31.5	23.0	$\bar{\sigma}_c = 8.00$ $K = 1.00$	ϵ	3.8	2.29^5	4.45^5	3.12	1.27	.287	10.4	2.25	4.02	3.54	1.38	.282	
$\overline{CIU}E$-1	30.1	24.6	$\bar{\sigma}_c = 4.00$ $K = 1.00$	σ	10.2	0.94	1.39^5	4.40	1.83	.235			same				Necking at $\epsilon = 2.5\%$
$\overline{CIU}E$-2	30.9	23.7^5	$\bar{\sigma}_c = 4.00$ $K = 1.00$	ϵ	1.0	1.05^5	2.31^5	2.68	1.30	.264	6.1	1.02^5	1.59^5	4.60	1.67	.256	
$\overline{CIU}E$-3	29.0	21.6	$\bar{\sigma}_c = 6.00$ $K = 1.00$	ϵ	7.7	1.41	2.20	4.56	1.85	.235			same				Necking at $\epsilon = 6.4\%$
$\overline{C(K_o)U}C$-1	31.2	25.9	$\bar{\sigma}_{1c} = 4.10$ $\bar{\sigma}_{3c} = 2.16$ $K = .528$	ϵ	0.3	1.35	3.04	2.59	0.62	.329	9.2	0.92	1.72	3.30	−13.6	.224	
$\overline{C(K_o)U}C$-2	29.7	23.3	$\bar{\sigma}_{1c} = 6.10$ $\bar{\sigma}_{3c} = 3.27$ $K = .536$	ϵ	0.3	2.00	4.64	2.51	0.54	.328	6.4	1.44^5	2.51^5	3.70	36.7	.237	
$C(K_o)$-$\overline{UU}C$-1	31.3	26.5	$\bar{\sigma}_{1c} = 4.08$ $\bar{\sigma}_{3c} = 2.16$ $K = .530$	ϵ	0.7	1.12	2.56	2.56	(3) 0.48	.275	9.7	0.78^5	1.43^5	3.42	(3) 1.19	.192	
$C(K_o)$-$\overline{UU}C$-2	31.0	24.9	$\bar{\sigma}_{1c} = 6.15$ $\bar{\sigma}_{3c} = 3.24$ $K = .528$	ϵ	0.7	1.74	3.94	2.58	(3) 0.51	.283	8.2	1.32	2.39	3.46	(3) 1.10	.215	
$\overline{C(K_o)U}RE$-1	30.1	26.1	$\bar{\sigma}_{1c} = 4.04$ $\bar{\sigma}_{3c} = 2.16$ $K = .535$	σ	16	0.55^5	0.58^5	22.4	1.36	.132			same				Severe necking at failure
$\overline{C(K_o)U}RE$-2	30.3^5	26.5	$\bar{\sigma}_{1c} = 4.03$ $\bar{\sigma}_{3c} = 2.16$ $K = .535$	σ	19	0.50^5	0.82^5	4.15	1.29	.125	14.2	0.48	0.77	4.32	1.32	.119	Necking at $\epsilon = 5.6\%$

Table 4.1 Continued

Test No.	Initial w (%)	Final w (%)	Consolidation Stresses [1]	Controlled	At $(\bar{\sigma}_1 - \bar{\sigma}_3)$ max. ϵ (%)	S_u	$\bar{p}$	$\bar{\sigma}_1/\bar{\sigma}_3$	A_f [3]	$S_u/\bar{\sigma}_{1c}$	At $(\bar{\sigma}_1/\bar{\sigma}_3)$ max. ϵ (%)	q	$\bar{p}$	$\bar{\sigma}_1/\bar{\sigma}_3$	A [2]	$q/\bar{\sigma}_{1c}$	Remarks
$C\overline{(K_o)}URE$-4	30.5	26.0	$\bar{\sigma}_{1c}=4.00$ $\bar{\sigma}_{3c}=2.16$ $K=.540$	ϵ	10.0	0.66	1.04	4.48	1.14	.165	9.7	0.65	1.02	4.52	1.15	.162	Necking at $\epsilon=6\%$
$C\overline{(K_o)}URE$-5	30.8	23.1	$\bar{\sigma}_{1c}=6.00$ $\bar{\sigma}_{3c}=3.24$ $K=.540$	ϵ	10.0	0.85^5	1.45^5	3.86	1.21	.142			same				Necking at $\epsilon=7.2\%$
$C\overline{(1/K_o)}UE$-1	29.6^5	23.0	$\bar{\sigma}_{1c}=4.00$ $\bar{\sigma}_{3c}=2.02$ $\frac{1}{K}=.505$	ϵ	0.2	1.31^5	2.64^5	2.98	1.06	.329	3.3	1.15	1.93	3.97	3.86	.288	Necking at $\epsilon=5\%$
$C\overline{(1/K_o)}UE$-3	30.6	22.8^5	$\bar{\sigma}_{1c}=6.00$ $\bar{\sigma}_{3c}=3.20$ $\frac{1}{K}=.533$	ϵ	0.17	1.74	3.82	2.68	1.65	.290	3.6	1.55	2.70	3.68	6.94	.258	Necking at $\epsilon=5\%$
$C\overline{(1/K_o)}URC$-2	30.2	25.0	$\bar{\sigma}_{1c}=4.00$ $\bar{\sigma}_{3c}=2.14$ $\frac{1}{K}=.535$	σ	9.8	1.09^5	1.92^5	3.64	0.78	.274	21	1.09	1.83	3.96	0.81	.273	Maybe leak during test
$C\overline{(1/K_o)}URC$-4	30.9	24.9	$\bar{\sigma}_{1c}=4.00$ $\bar{\sigma}_{3c}=2.13$ $\frac{1}{K}=.533$	σ	8.3	1.05^5	1.93^5	3.40	0.78	.264	18.2	0.97	1.77	3.43	0.84	.252	
$C\overline{(1/K_o)}URC$-5	29.7	22.8	$\bar{\sigma}_{1c}=4.00$ $\bar{\sigma}_{3c}=1.99$ $\frac{1}{K}=.497$	ϵ	13.7	1.32^5	2.24^5	3.88	0.66	.332	9.4	1.26^5	2.14^5	3.88	0.69	.316	
$C\overline{(1/K_o)}URC$-6	31.1	22.4	$\bar{\sigma}_{1c}=6.00$ $\bar{\sigma}_{3c}=2.90$ $\frac{1}{K}=.483$	ϵ	11.9	2.07	3.25	4.50	0.66	.345			same				

(1) Full correction for filler strips applied at end of K_o consolidation (where applicable)

(2) $A = (\Delta u - \Delta\sigma_r)/(\Delta\sigma_a - \Delta\sigma_r)$ for all tests failed in compression.

$A = (\Delta u - \Delta\sigma_a)/(\Delta\sigma_r - \Delta\sigma_a)$ for all tests failed in extension.

(3) A based on Δu and $\Delta(\sigma_1 - \sigma_3)$ from point where $(\sigma_1 - \sigma_3) = 0$

TABLE 4.2

MISCELLANEOUS DATA FOR CU TRIAXIAL TESTS ON N.C. BOSTON BLUE CLAY

Stresses in kg/cm^2

Test No.	Initial w (%)	Final w (%)	Final $(\frac{\Delta e}{1+e_o})$ (%)	Time* Consolidation (days)	Consolidation History	Time (hrs.) To ϵ = 1%	Time (hrs.) To ϵ_f (ϵ_f %)**	Controlled ϵ or σ
$\overline{\text{CIUC}}$-2	31.4	25.4	11.5	4	$\bar{\sigma}_c$ = 1.0, 2.0, 4.0	3	4 (2.8)	ϵ
$\overline{\text{CIUC}}$-1	30.4	23.45	12.5	3	$\bar{\sigma}_c$ = 1.0, 2.0, 4.0, 6.0	1.5	4 (3.5)	ϵ
$\overline{\text{CIUC}}$-3	31.5	23.0	14.8	3	$\bar{\sigma}_c$ = 1.0, 2.0, 4.0, 8.0	~2	- (3.8)	ϵ
$\overline{\text{CIUE}}$-1	30.1	24.6	10.7	3	$\bar{\sigma}_c$ = 1.0, 2.0, 4.0	8	10 (10.2)	σ
$\overline{\text{CIUE}}$-2	30.9	23.75	13.1	12	$\bar{\sigma}_c$ = 1.36, 3.0, 4.0	23	23 (1.0)	ϵ
$\overline{\text{CIUE}}$-3	29.0	21.6	14.1	6	$\bar{\sigma}_c$ = 1.0, 2.0, 4.0, 6.0	~30	80 (7.7)	ϵ
$\overline{\text{C(K}_o\text{)UC}}$-1	31.2	25.9	10.0	6	$\bar{\sigma}_{1c}$ = 1.12, 1.5, 2.17, 2.72, 3.27, 3.61, 4.10 K = .528	1	0.5 (0.3)	ϵ
$\overline{\text{C(K}_o\text{)UC}}$-2	29.7	23.3	11.4	8	$\bar{\sigma}_{1c}$ = 1.0, 1.2, 1.5, 2.0, 2.6, 3.15, 3.8, 4.44, 5.1, 5.56, 6.10 K = .536	1	0.7 (0.3)	ϵ
C(K$_o$)-$\overline{\text{UUC}}$-1	31.3	26.5	8.4	8	$\bar{\sigma}_{1c}$ = 1.12, 1.5, 2.17, 2.72, 3.27, 3.61, 4.08 K = .530	1.2	1 (0.7)	ϵ
C(K$_o$)-$\overline{\text{UUC}}$-2	31.0	24.9	12.5	7	$\bar{\sigma}_{1c}$ = 1.12, 1.5, 2.72, 3.28, 3.61, 4.0, 4.66, 5.32, 6.15 K = .528	2.2	2 (0.7)	ϵ
$\overline{\text{C(K}_o\text{)URE}}$-1	30.1	26.1	6.6	3	$\bar{\sigma}_{1c}$ = 1.12, 1.5, 2.17, 2.7, 3.27, 3.61, 4.04 K = .535	8	10 (16)	σ
$\overline{\text{C(K}_o\text{)URE}}$-2	30.55	26.5	7.5	3	$\bar{\sigma}_{1c}$ = 1.12, 1.5, 2.17, 2.7, 3.27, 3.61, 4.03 K = .535	7.5	13 (19)	σ
$\overline{\text{C(K}_o\text{)URE}}$-4	30.5	26.0	8.0	6	$\bar{\sigma}_{1c}$ = 1.12, 1.5, 2.17, 2.7, 3.27, 3.61, 4.00 K = .540	26	120 (10)	ϵ
$\overline{\text{C(K}_o\text{)URE}}$-5	30.8	23.1	12.7	4	$\bar{\sigma}_{1c}$ = 1.12, 1.5, 2.17, 2.7, 3.27, 3.61, 4.16, 4.76, 5.37, 6.00 K = .540	26	120 (10)	ϵ
$\overline{\text{C(1/K}_o\text{)UE}}$-1	29.65	23.0	11.8	6	$\bar{\sigma}_{1c}$ = 0.6, 0.8, 1.1, 1.5, 2.0, 2.5, 3.0, 3.5, 4.00 1/K = .505	14	7.5 (0.2)	ϵ
$\overline{\text{C(1/K}_o\text{)UE}}$-3	30.6	22.85	13.6	4	$\bar{\sigma}_{1c}$ = 0.6, 0.8, 1.1, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5 5.0, 5.5, 6.0 1/K = .533	19	10 (0.15)	ϵ
$\overline{\text{C(1/K}_o\text{)URC}}$-2	30.2	25.0	11.5	2	$\bar{\sigma}_{1c}$ = 0.6, 0.8, 1.1, 1.5, 2.0, 2.5, 3.0, 3.5, 4.00 1/K = .535	80	290 (9.8)	σ
$\overline{\text{C(1/K}_o\text{)URC}}$-4	30.9	24.9	11.6	3	$\bar{\sigma}_{1c}$ = 0.6, 0.8, 1.1, 1.5, 2.0, 2.5, 3.0, 3.5, 4.00 1/K = .533	110	240 (8.3)	σ
$\overline{\text{C(1/K}_o\text{)URC}}$-5	29.7	22.8	11.3	4	$\bar{\sigma}_{1c}$ = 0.6, 0.8, 1.1, 1.5, 2.0, 2.5, 3.0, 3.5, 4.00 1/K = .497	12	76 (13.7)	ϵ
$\overline{\text{C(1/K}_o\text{)URC}}$-6	31.1	22.4	15.4	2	$\bar{\sigma}_{1c}$ = 0.6, 0.8, 1.1, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.00 1/K = .483	19	75 (11.9)	ϵ

* For final consolidation increment

** Value of ϵ_f in ( ).

Table 4.3 Average Strength Parameters from $\overline{CU}$ Triaxial Tests on N.C. Boston Blue Clay (Data from strain controlled tests at $\bar{\sigma}_{1c} = 4$ and 6 averaged unless otherwise noted. $\bar{c}$ assumed equal to zero)

Type of Test	K	At $(\sigma_1 - \sigma_3)_{max.}$ (1)				At $(\bar{\sigma}_1 / \bar{\sigma}_3)_{max.}$ (1)				Remarks
		ϵ(%)	$S_u/\bar{\sigma}_{1c}$	A_f	$\bar{\Phi}_u$°	ϵ(%)	$q/\bar{\sigma}_{1c}$	A	$\bar{\phi}$°	
$\overline{CIUC}$	1.00	3.4 (2.8-3.8)	0.285 (.26-.31)	1.25 (1.1-1.4)	29.5 (28.5-31)	10.5 (10-11)	0.275 (.25-.29)	1.45 (1.3-1.6)	33.5 (33-34)	Ave. from 3 tests with $\bar{\sigma}_c$ = 4, 6 and 8
$\overline{CIUE}$	1.00	6 (1-10)	0.245 (.235-.265)	1.7 (1.3-1.85)	36.5 (27-40)	8 (6-10)	0.24 (.235-.255)	1.8 (1.65-1.85)	39.5 (39-40)	Ave. includes 1 σ contr. test. Problem with necking
$\overline{C(K_o)UC}$	0.53	0.3 (same)	0.33 (same)	0.58 (.54-.62)	26.0 (25.5-26.5)	8 (6.5-9)	0.23 (.225-.235)	>35 or negative	33.5 (32-35)	
$C(K_o) \cdot \bar{U}\bar{U}C$	0.53	0.7 (same)	0.28 (.275-.285)	0.50 (.48-.51)	26.0 (same)	9 (8-10)	0.205 (.19-.215)	1.15 (1.1-1.2)	33.5 (same)	A calculated from Δu and $\Delta(\sigma_1 - \sigma_3)$ after perfect sampling
$\overline{C(K_o)URE}$	0.54	10 (same)	0.155 (.14-.165)	1.17 (1.15-1.2)	38.0 (36-39.5)	10 (same)	0.15 (.14-.16)	1.18 (1.15-1.2)	38.0 (36-39.5)	Problem with necking at ϵ > 6-7% $A = \frac{\Delta u - \Delta \sigma_a}{\Delta \sigma_r - \Delta \sigma_a}$.
$\overline{C(1/K_o)UE}$	1.93	0.2 (.15-.2)	0.31 (.29-.33)	1.35 (1.05-1.65)	28.5 (27-30)	3.5 (3.3-3.6)	0.275 (.26-.29)	5.4 (3.9-6.9)	36.0 (35-37)	Promblem with necking at ϵ - 5%.
$\overline{C(1/K_o)URC}$	2.04	13 (12-14)	0.34 (.33-.345)	0.66 (same)	38.0 (36.5-39.5)	11 (9.4-12)	0.33 (.315-.345)	0.67 (.66-.69)	38.0 (36.5-39.5)	σ contr. tests showed 20% lower S_u. $A = \frac{\Delta u - \Delta \sigma_r}{\Delta \sigma_a - \Delta \sigma_r}$

(1) Approximate range of values shown in ( ).

Fig. 4.1 Effective Stress Paths for $\overline{CU}$ Tests on N.C. BBC with Isotropic Consolidation

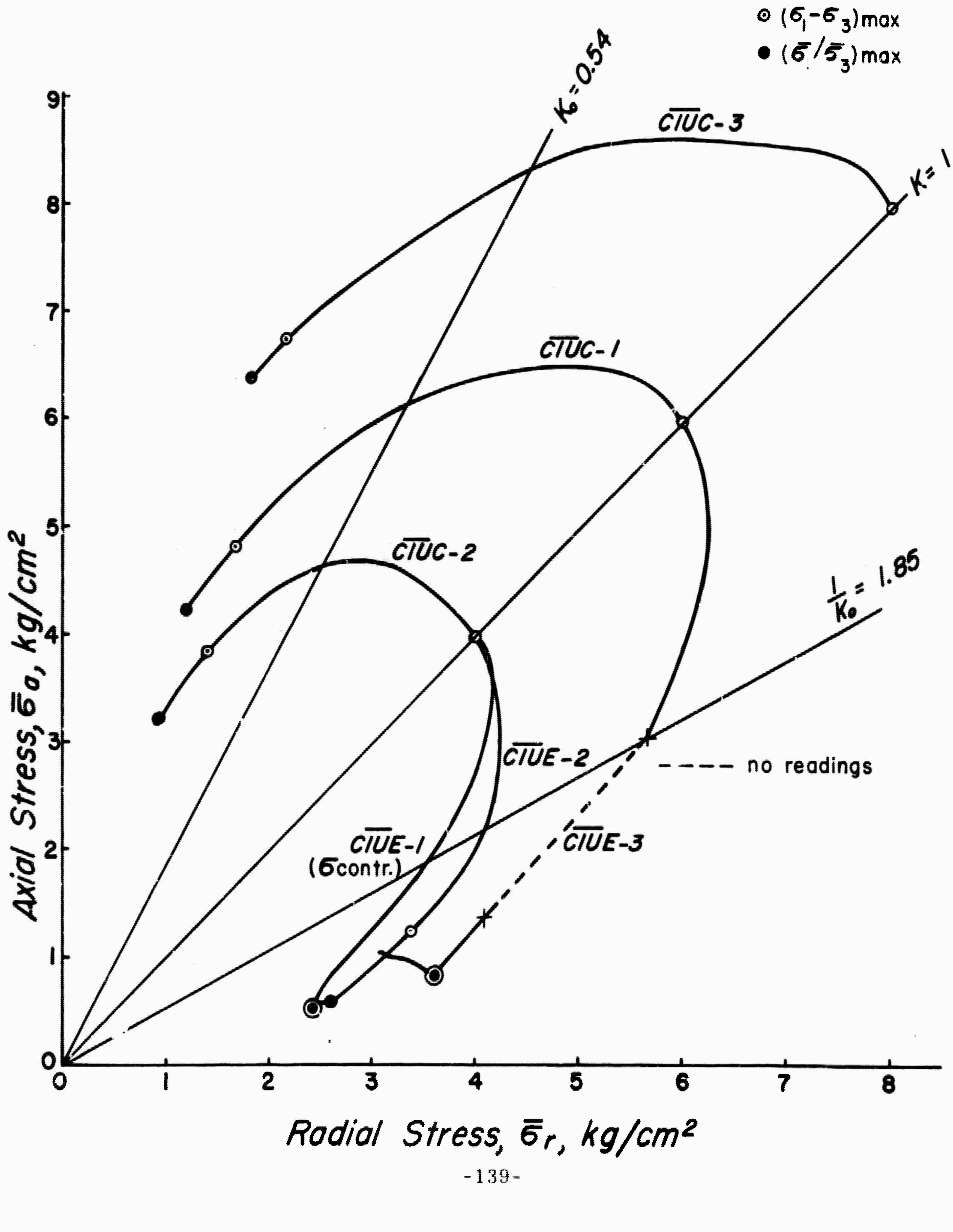


Fig. 4.2 Effective Stress Paths for $\overline{CU}$ Tests on N.C. BBC with K_0 Consolidation

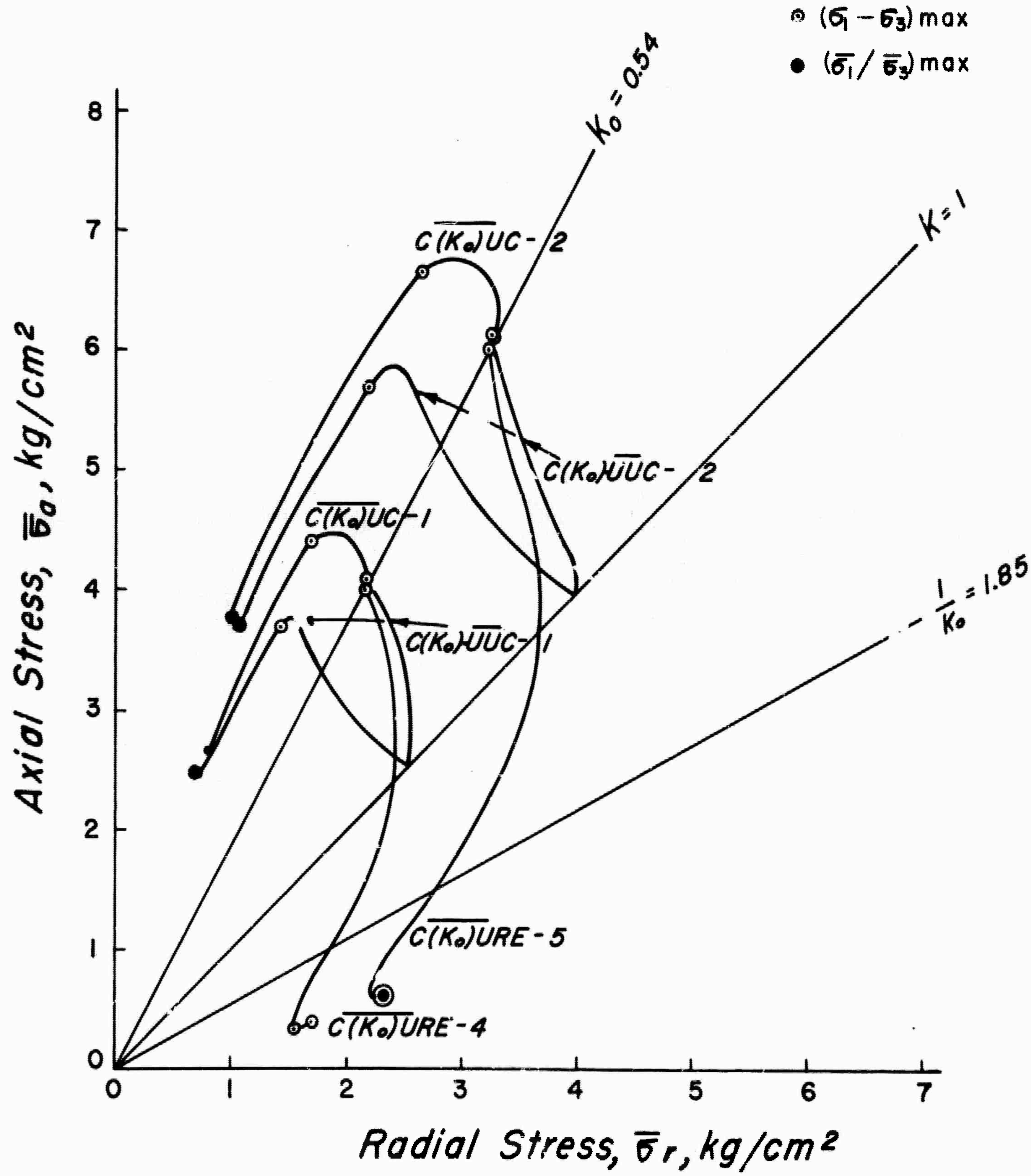


Fig. 4.3 Effective Stress Paths for $\overline{CU}$ Tests on N.C. BBC with $\frac{1}{K_o}$ Consolidation

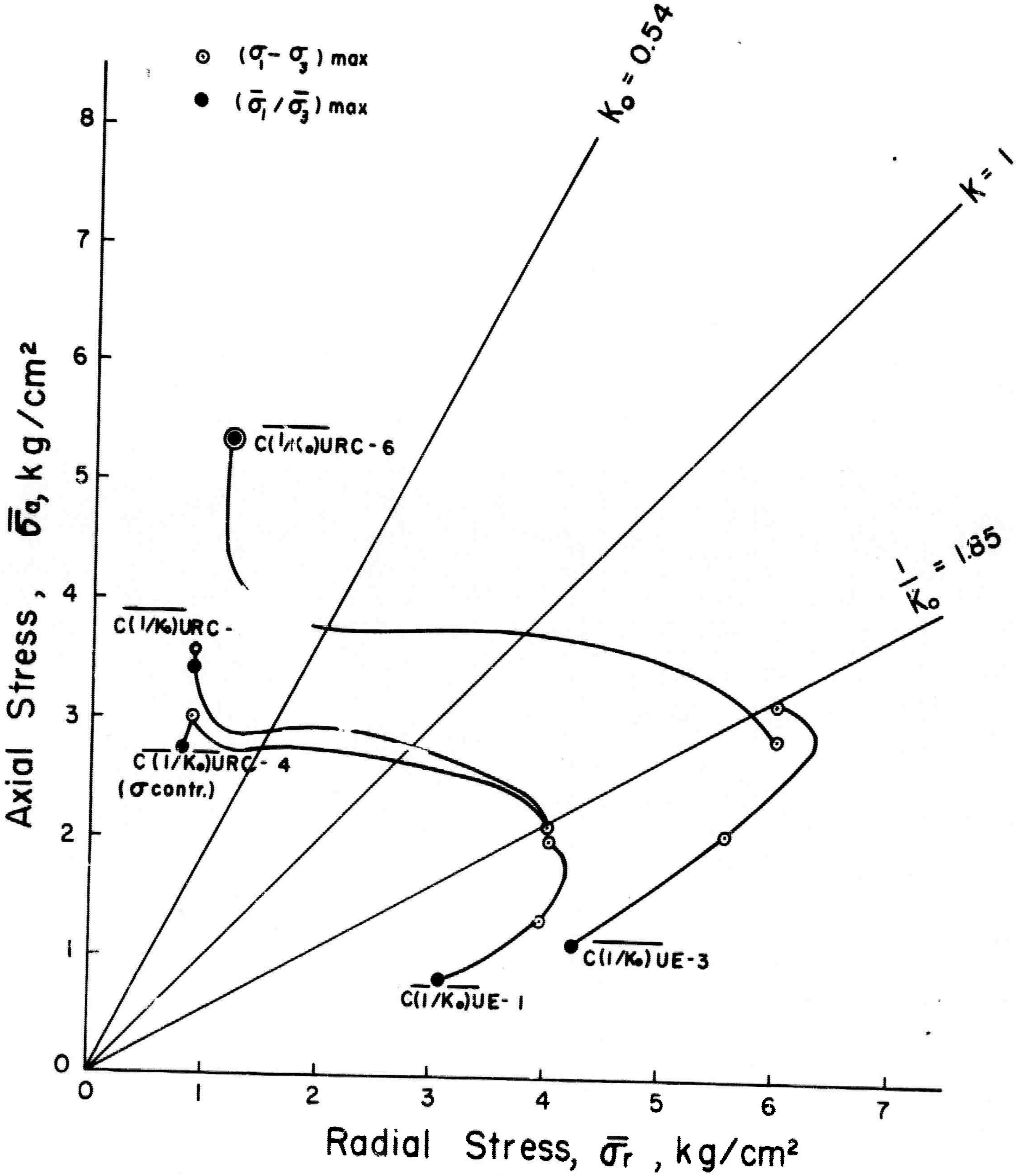


Fig. 4.4 Effective Stress Paths for $\overline{CU}$ Tests on N.C. BBC with Stress Controlled Shear

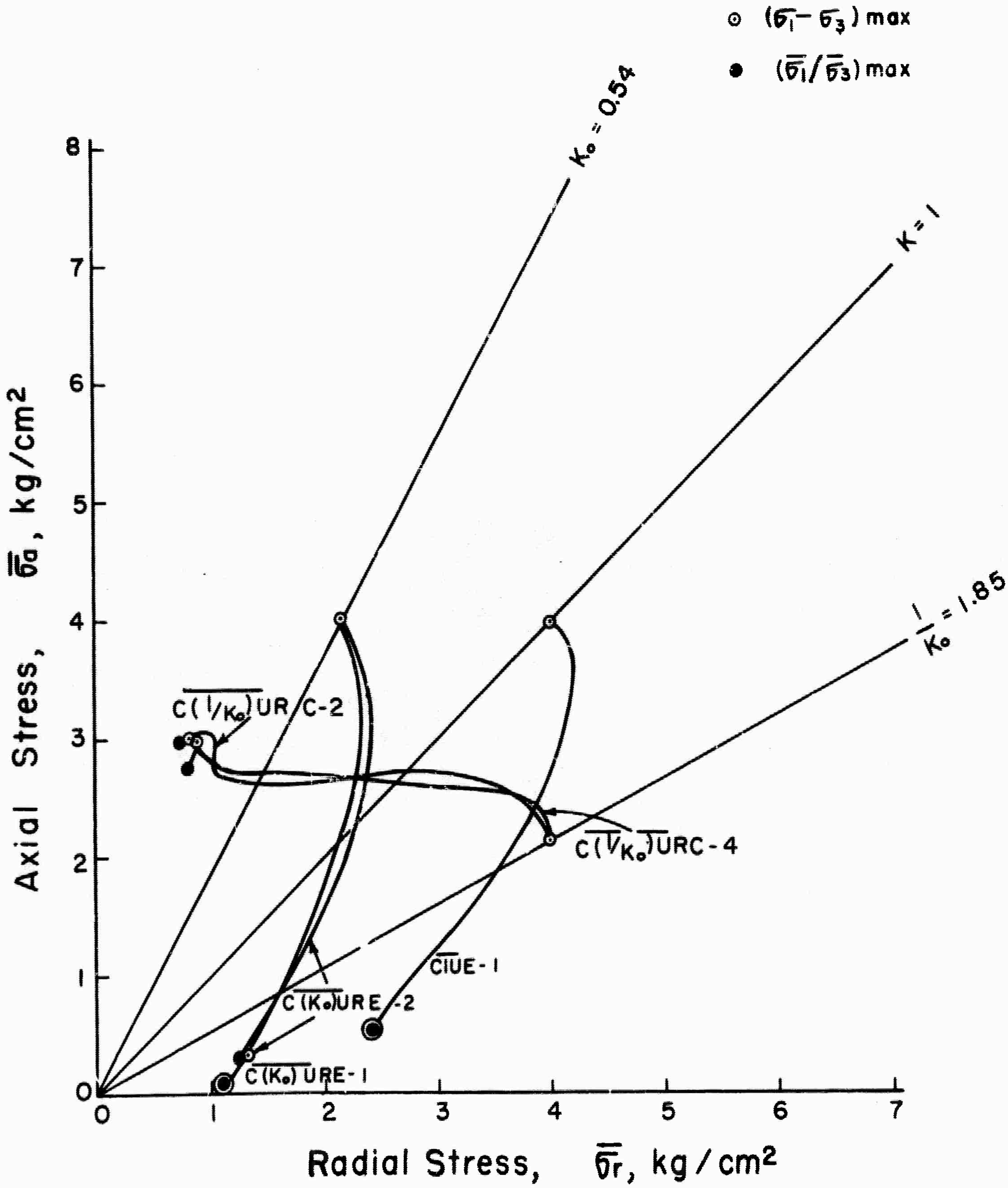


Fig. 4.5 Summary of Strength Data from $\overline{CU}$ Tests on N.C. BBC, Isotropic Consolidation

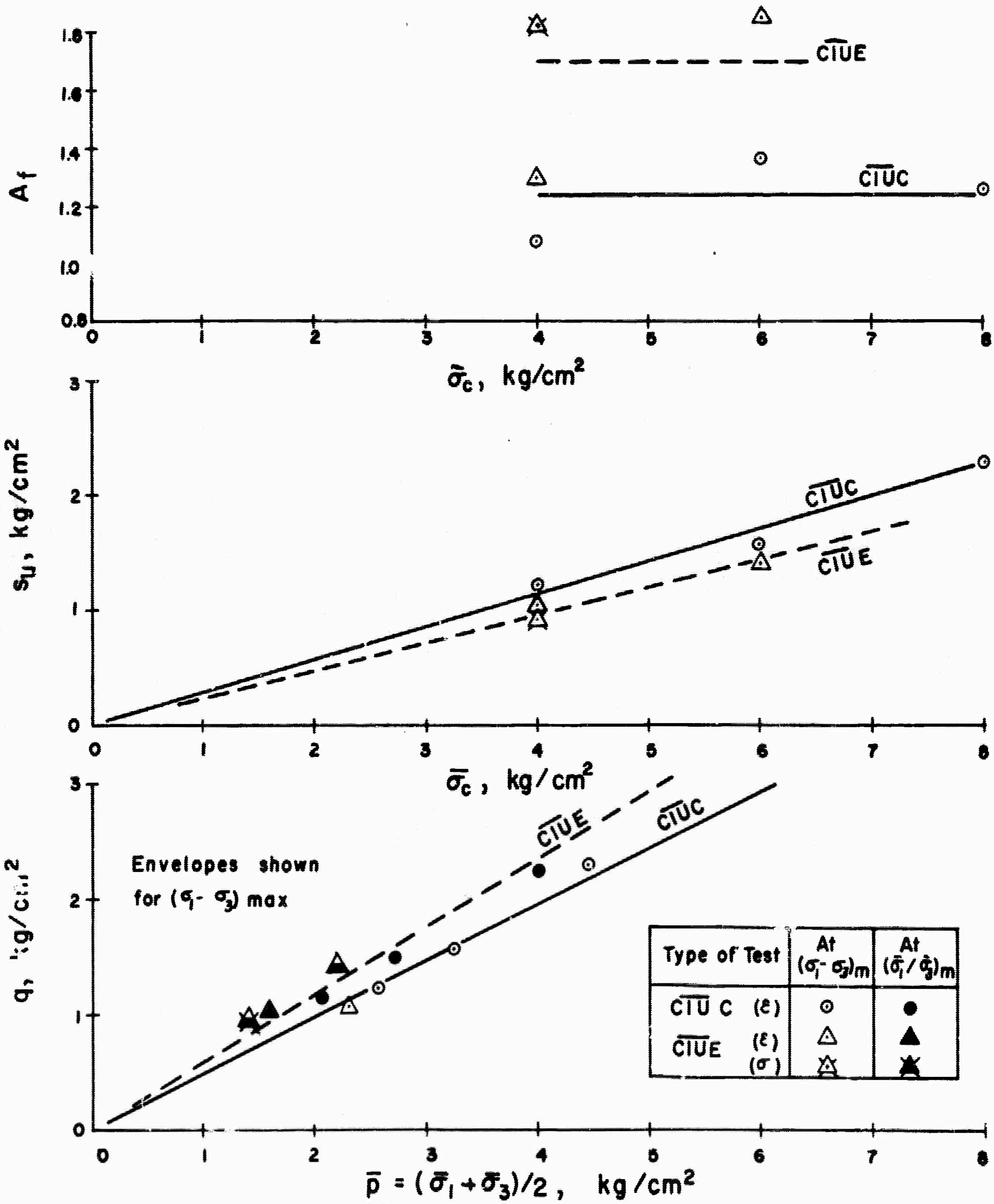


Fig. 4.6 Summary of Strength Data from $\overline{CU}$ Tests on N.C. BBC with K_o Consolidation

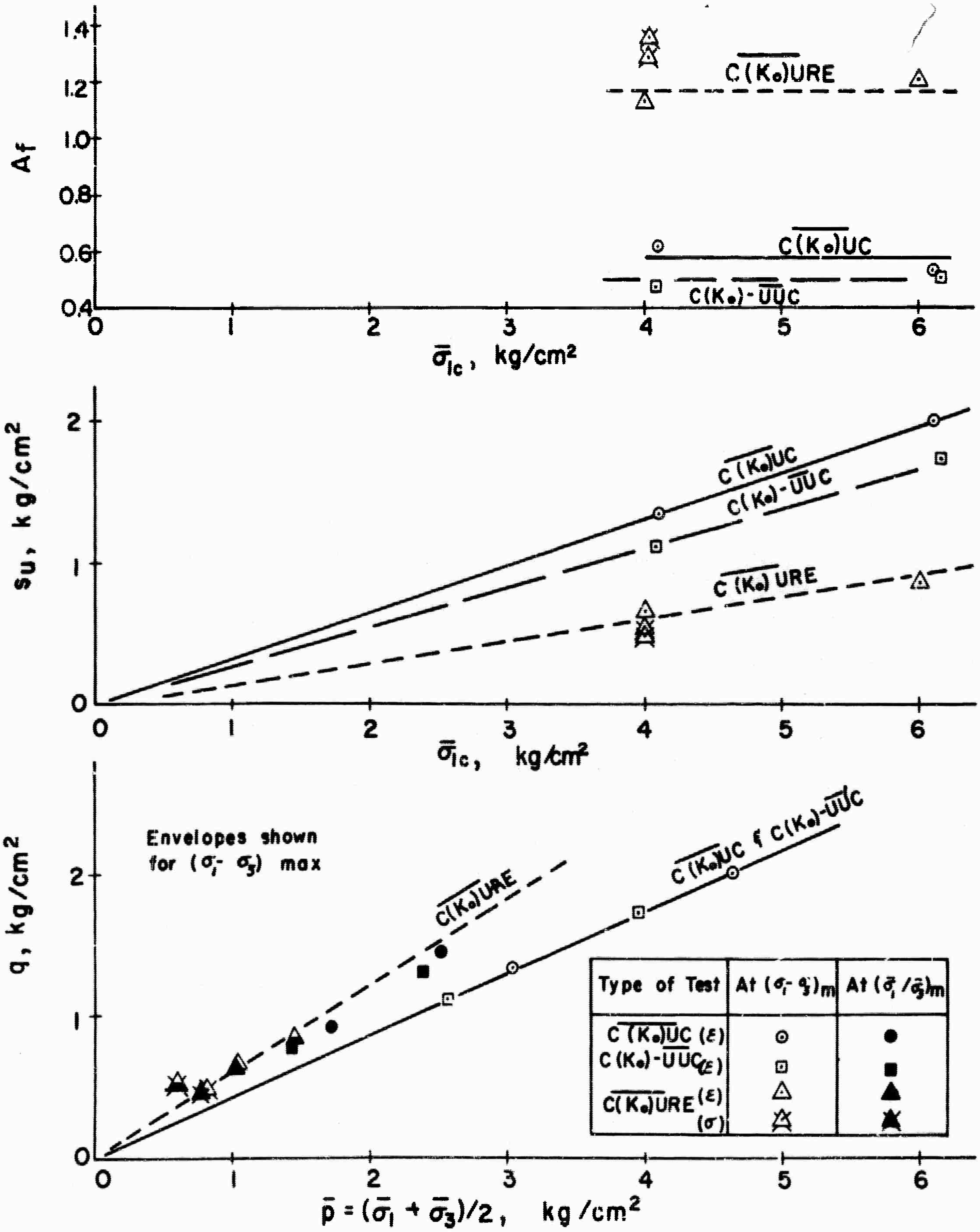


Fig. 4.7 Summary of Strength Data from $\overline{CU}$ Tests on N.C. BBC with $1/K_o$ Consolidatior

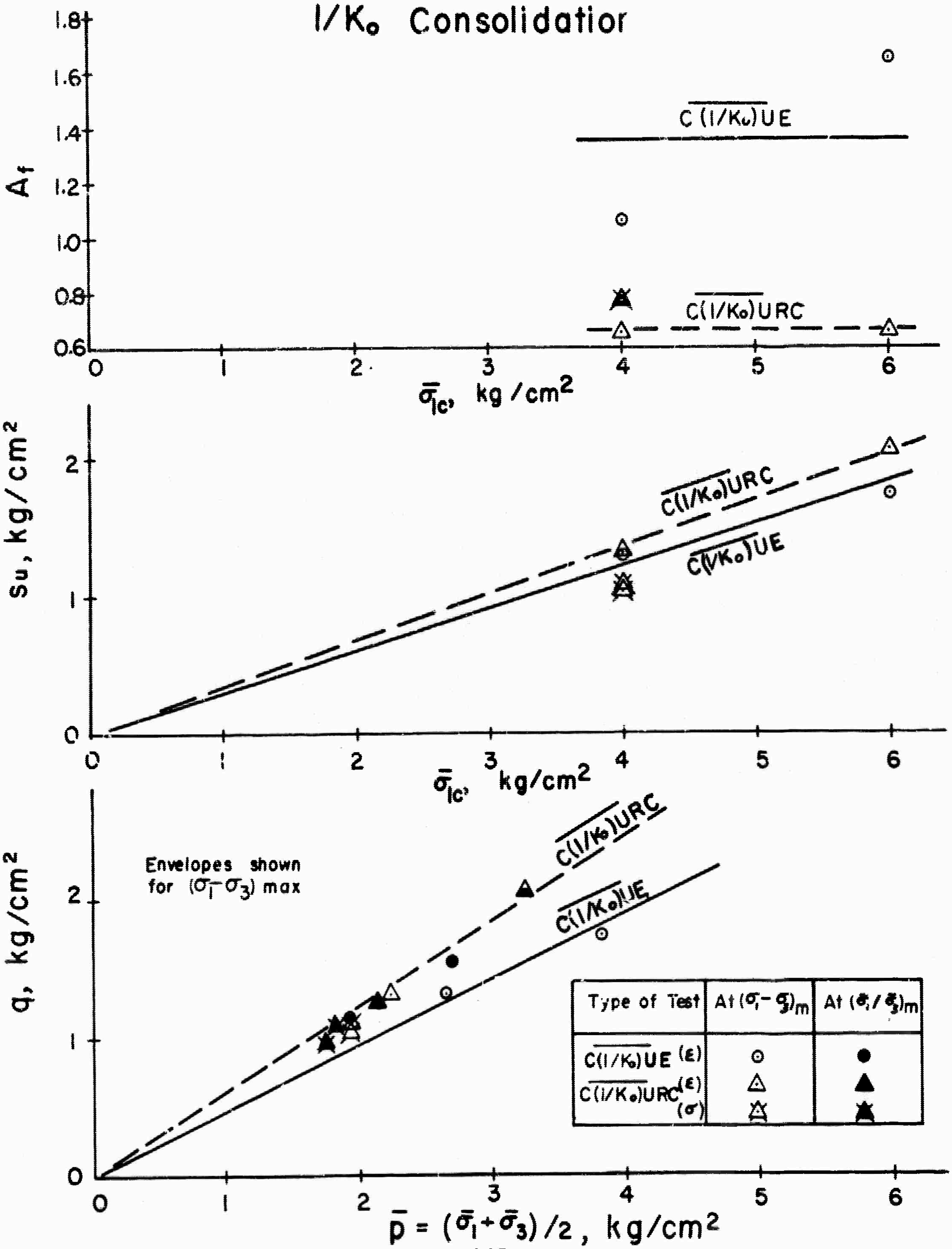


Fig. 4.8 Effect of Water Content at Failure on Undrained Strength Behavior of N.C. BBC
(Data from various batches over 3 year period, but $\bar{\sigma}_{lc}$ = 6 ± 0.1 kg/cm² for all tests.)

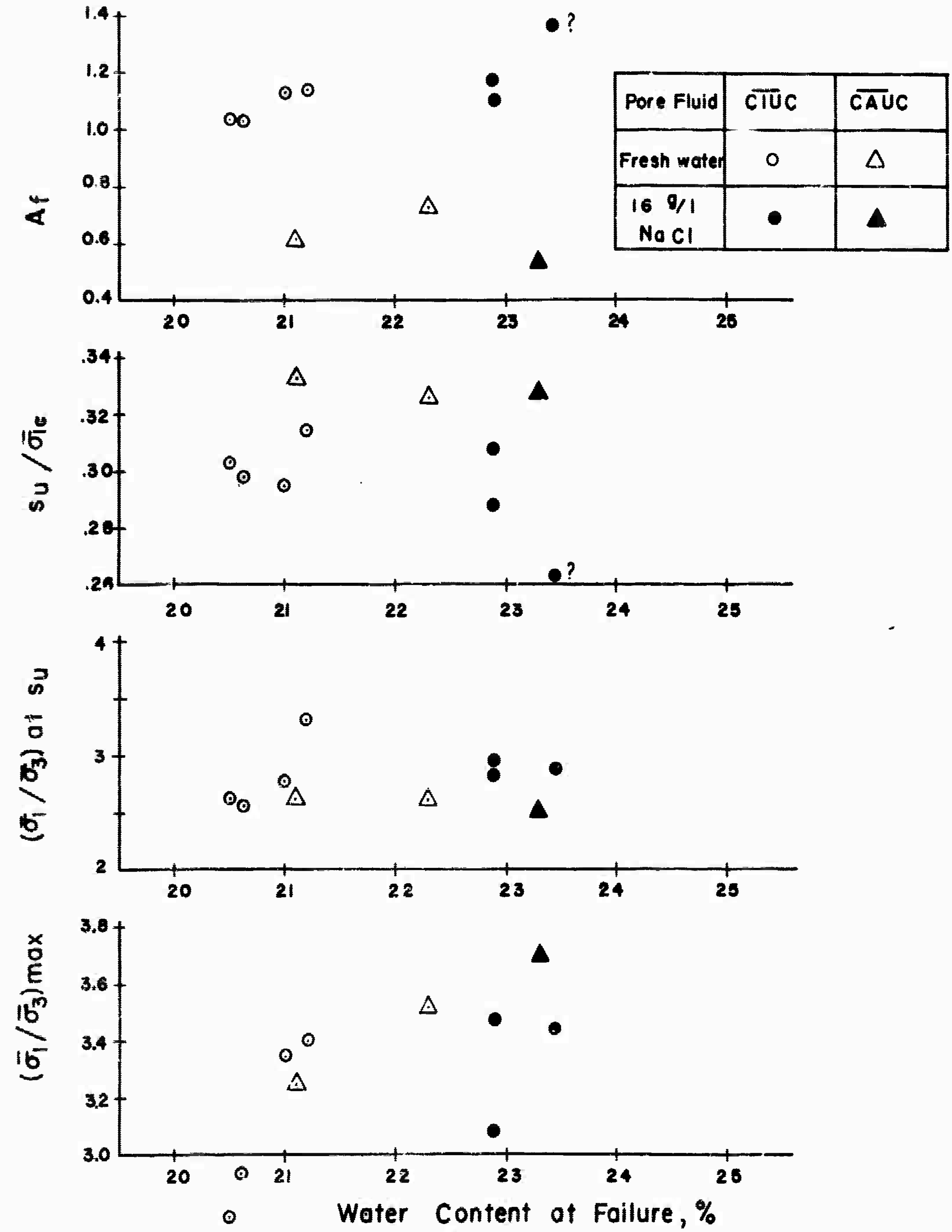
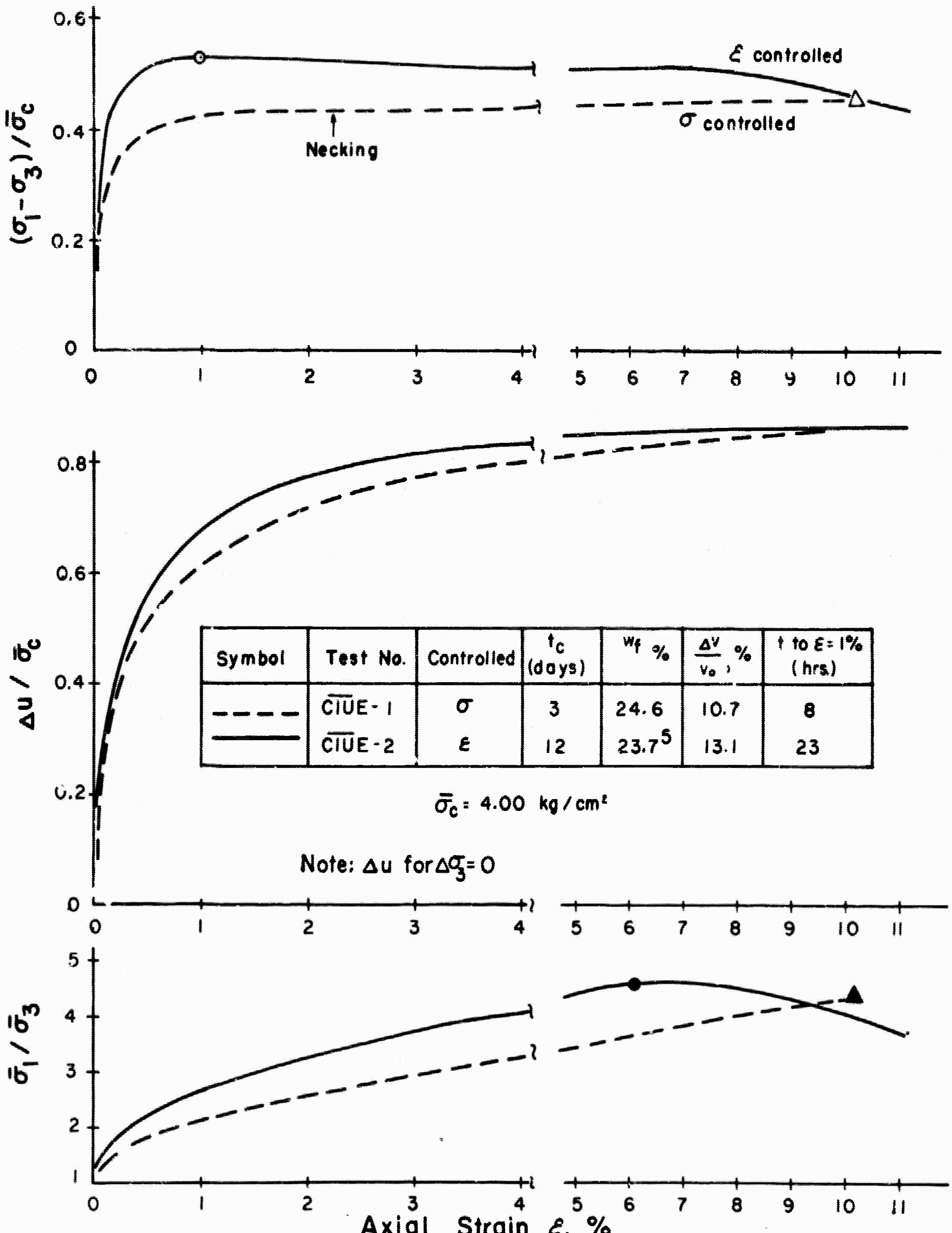


Fig. 4.9 Comparison of Stress and Strain Controlled $\overline{\text{CIUE}}$ Tests on N.C. BBC



Symbol	Test No.	Controlled	t_c (days)	w_f %	$\frac{\Delta V}{V_o}$, %	t to $\mathcal{E}$ = 1% (hrs.)
– – – –	$\overline{\text{CIUE}}$-1	σ	3	24.6	10.7	8
——	$\overline{\text{CIUE}}$-2	$\mathcal{E}$	12	23.7[5]	13.1	23

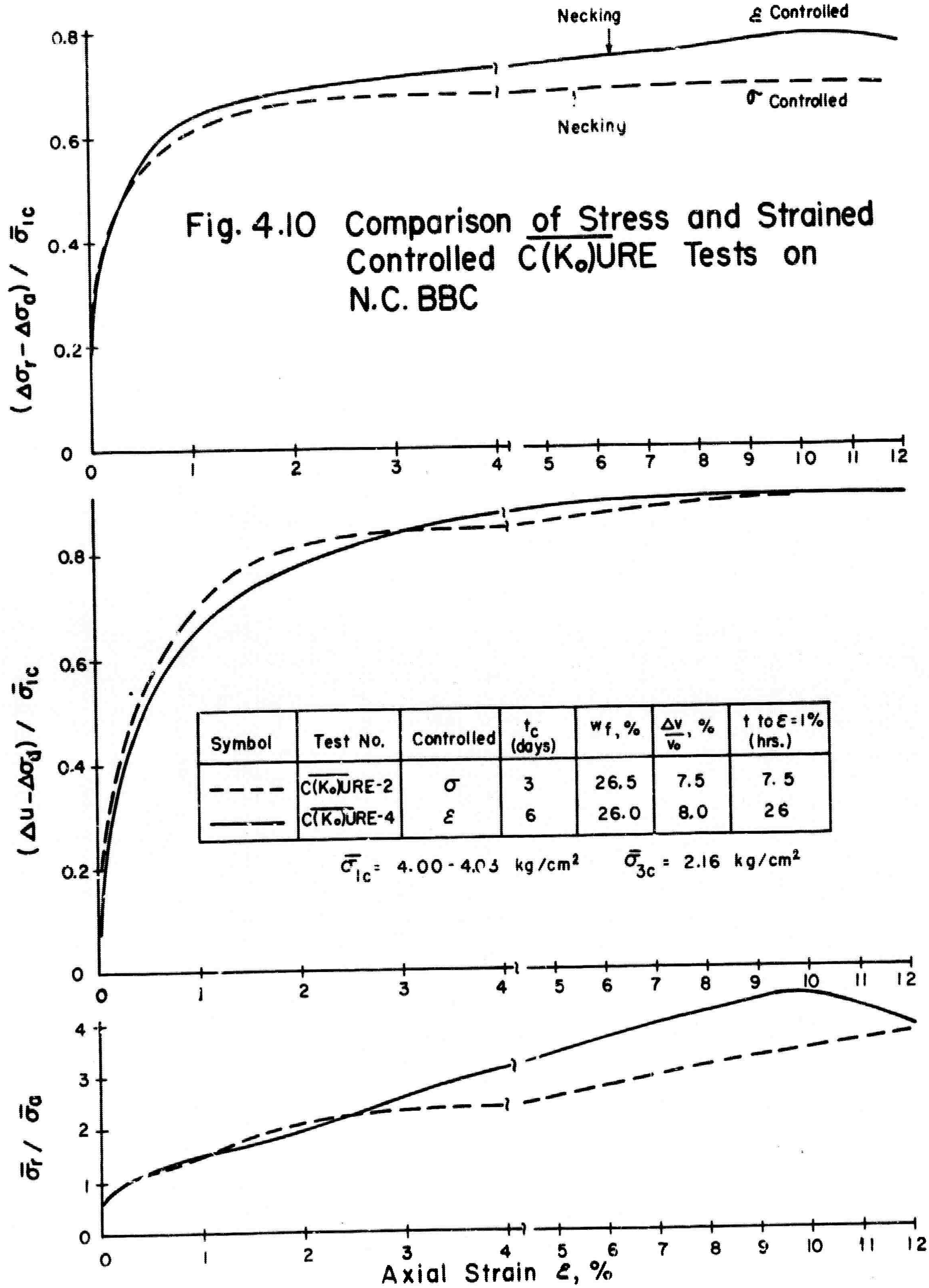
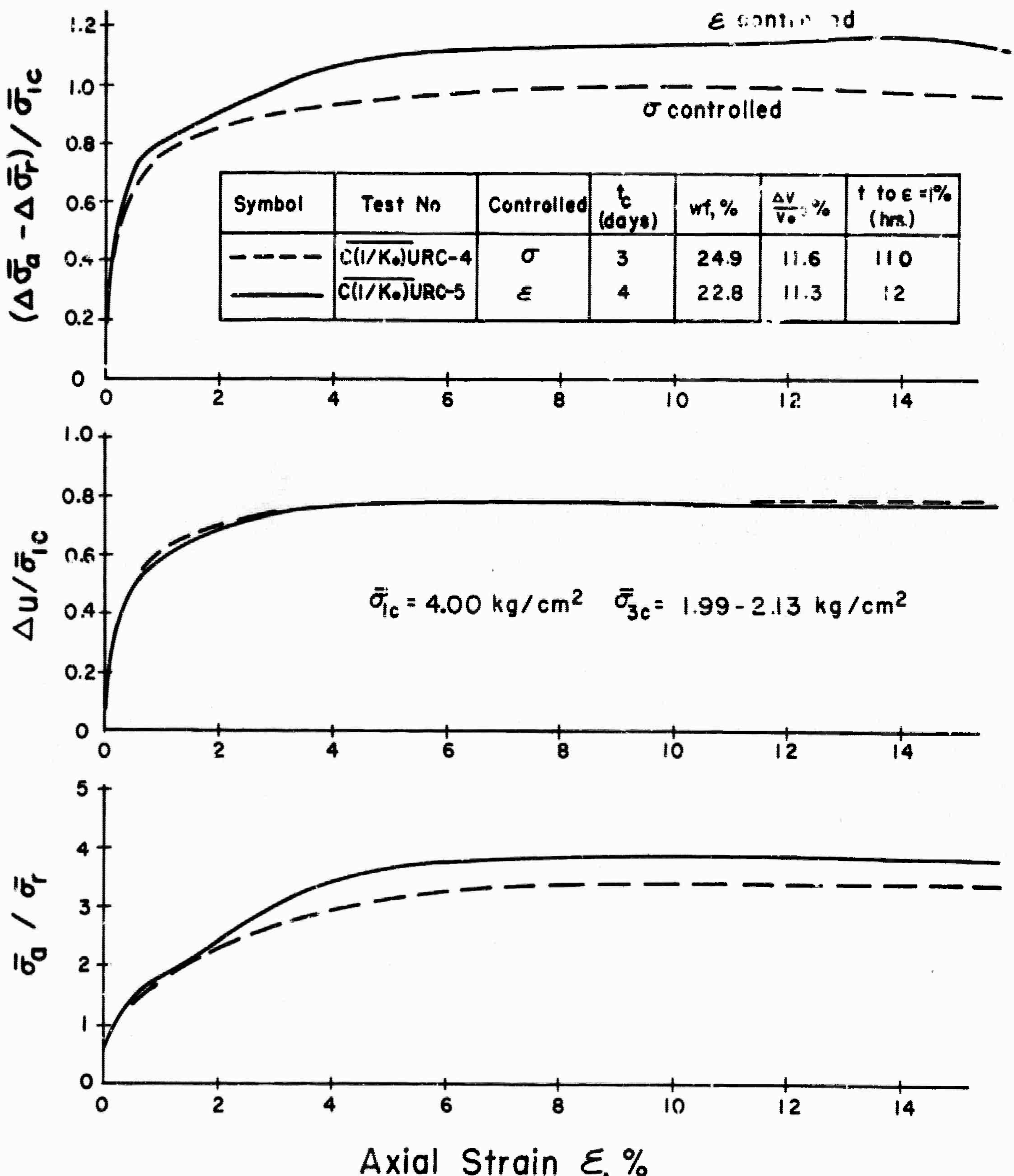


Fig. 4.10 Comparison of Stress and Strained Controlled C(K₀)URE Tests on N.C. BBC

Symbol	Test No.	Controlled	t_c (days)	w_f, %	$\frac{\Delta V}{V_0}$, %	t to ε=1% (hrs.)
– – – –	$\overline{C(K_o)}$URE-2	σ	3	26.5	7.5	7.5
———	$\overline{C(K_o)}$URE-4	ε	6	26.0	8.0	26

Fig. 4.11 Comparison of Stress and Strain Controlled $\overline{C(1/K_o)}URC$ Tests on N.C. BBC



Symbol	Test No	Controlled	t_c (days)	wf, %	$\frac{\Delta V}{V_o}$, %	t to ε = 1% (hrs.)
- - - -	$\overline{C(1/K_o)}URC$-4	σ	3	24.9	11.6	110
———	$\overline{C(1/K_o)}URC$-5	ε	4	22.8	11.3	12

Fig. 4.12 Effective Stress Paths from $\overline{CIU}$ Compression and Extension Tests on N.C. BBC

Note: All tests were strain controlled.

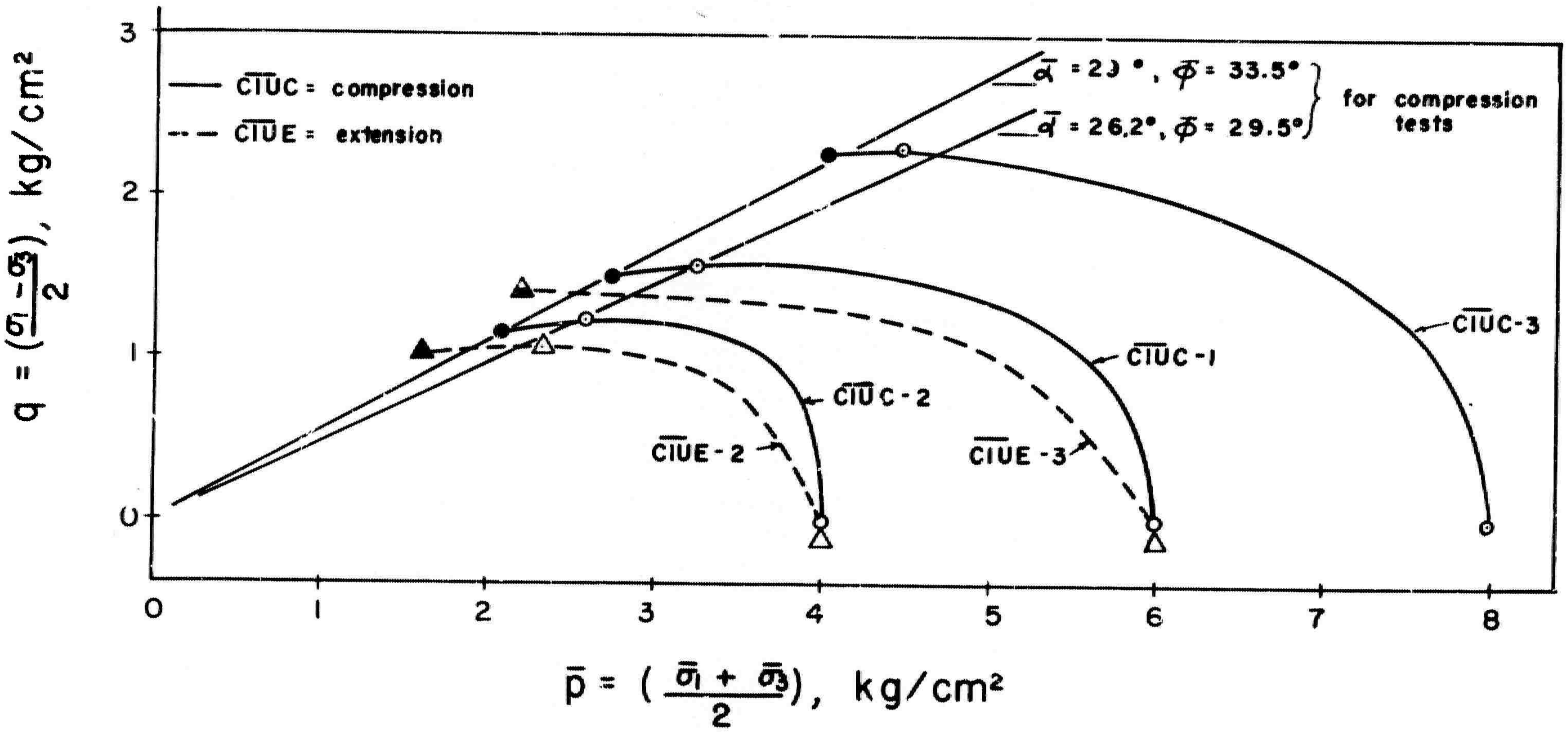


Fig. 4.13 Comparison of $\overline{CIUC}$ and $\overline{CIUE}$ Tests on N.C. BBC

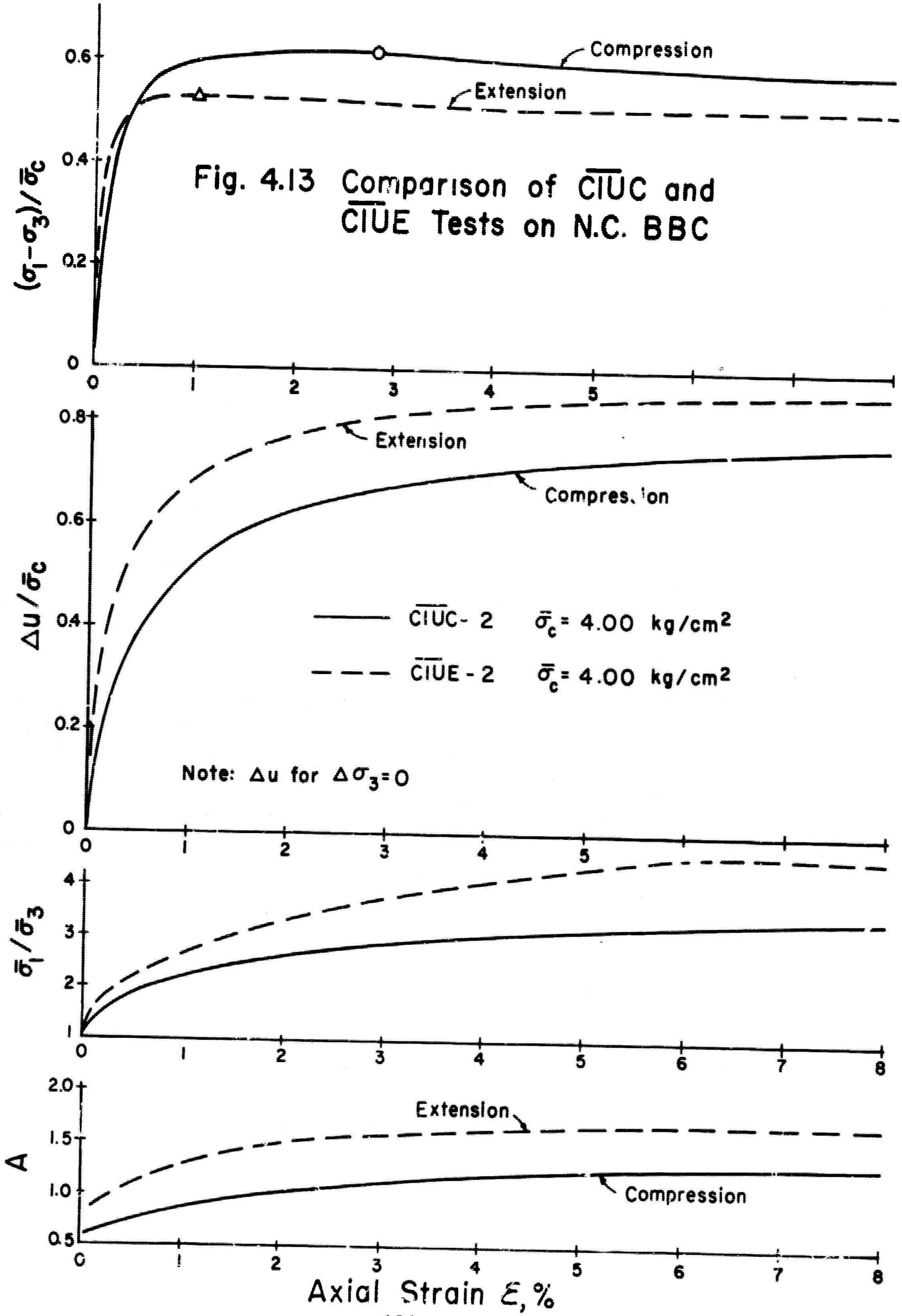


Fig. 4.14 Effective Stress Paths from $\overline{\text{CAU}}$ Compression and Extension Tests on N.C. BBC

Note: All tests were strain controlled.

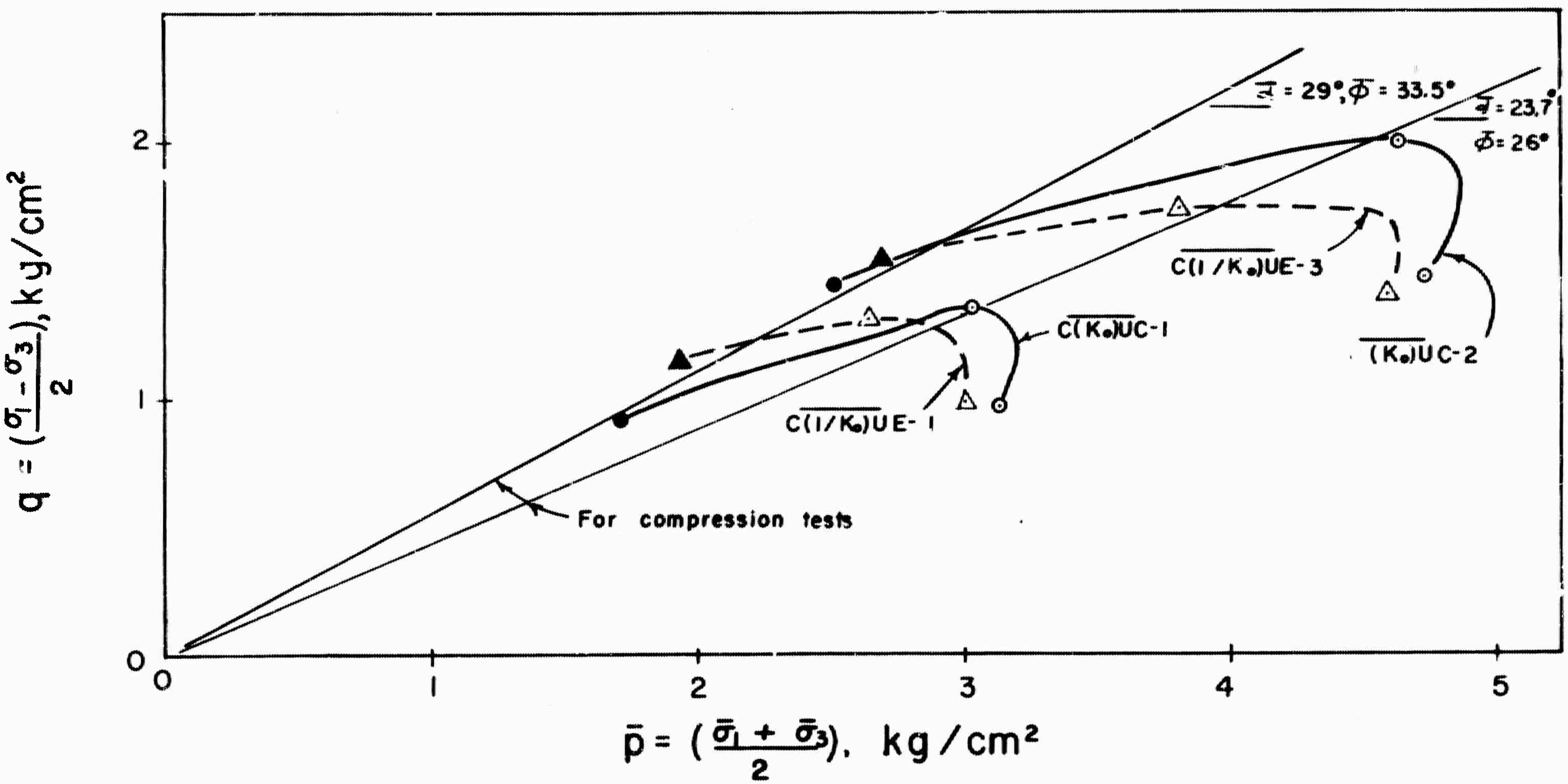
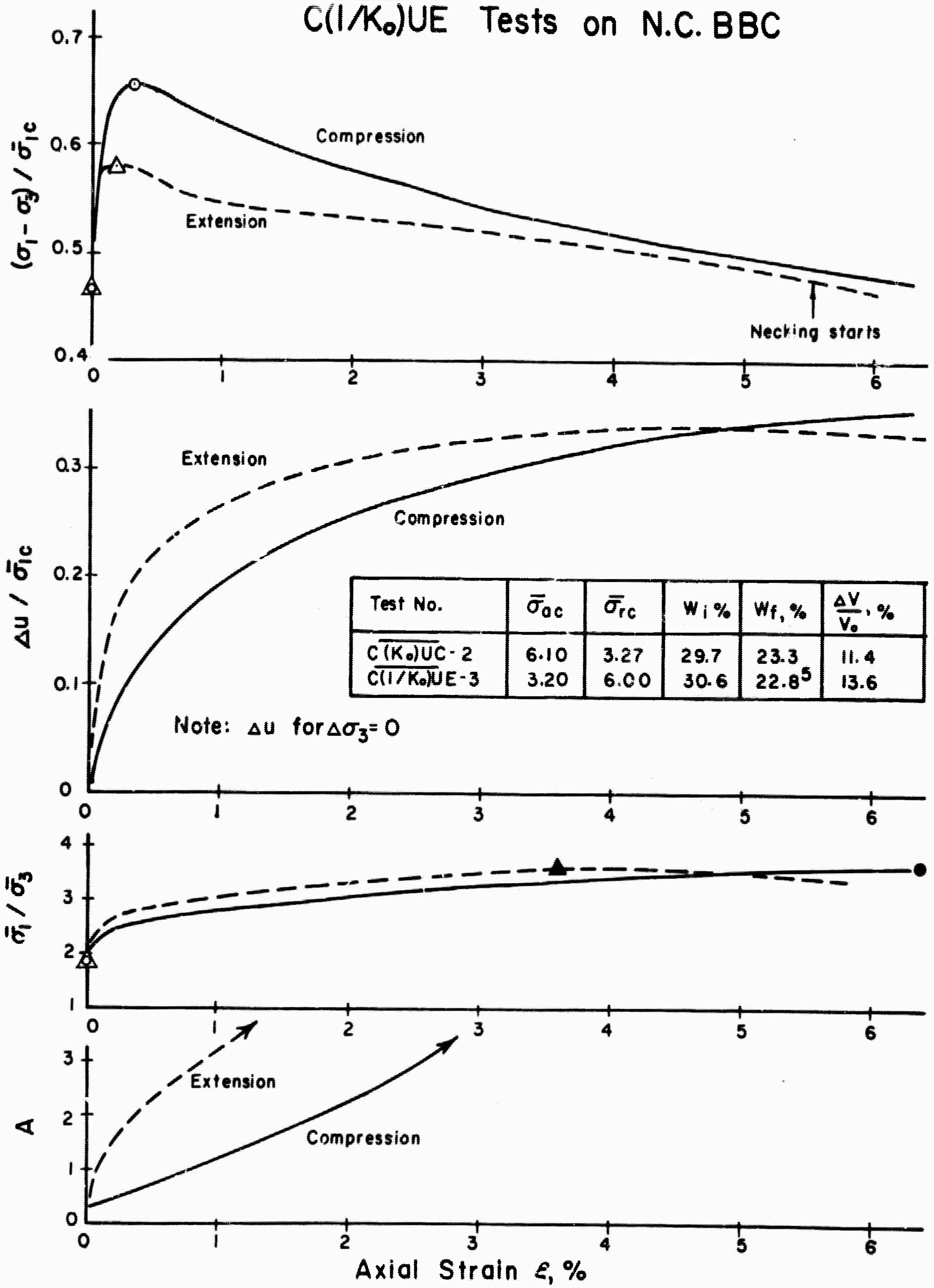


Fig. 4.15 Comparison of $\overline{C(K_o)UC}$ and $\overline{C(1/K_o)UE}$ Tests on N.C. BBC



Test No.	$\bar{\sigma}_{ac}$	$\bar{\sigma}_{rc}$	W_i %	W_f, %	$\frac{\Delta V}{V_o}$, %
$\overline{C(K_o)UC}$-2	6.10	3.27	29.7	23.3	11.4
$\overline{C(1/K_o)UE}$-3	3.20	6.00	30.6	22.8^5	13.6

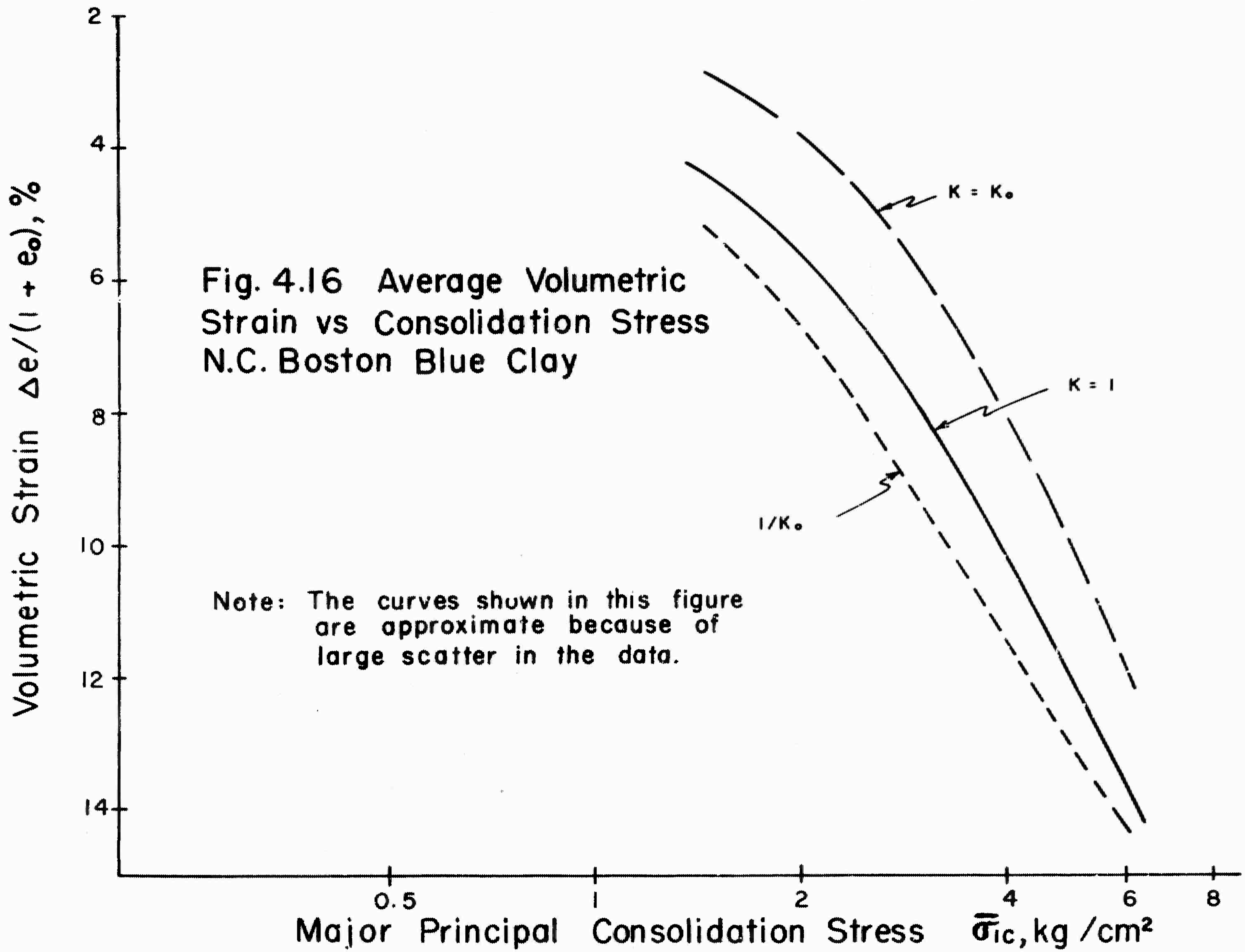


Fig. 4.16 Average Volumetric Strain vs Consolidation Stress N.C. Boston Blue Clay

Note: The curves shown in this figure are approximate because of large scatter in the data.

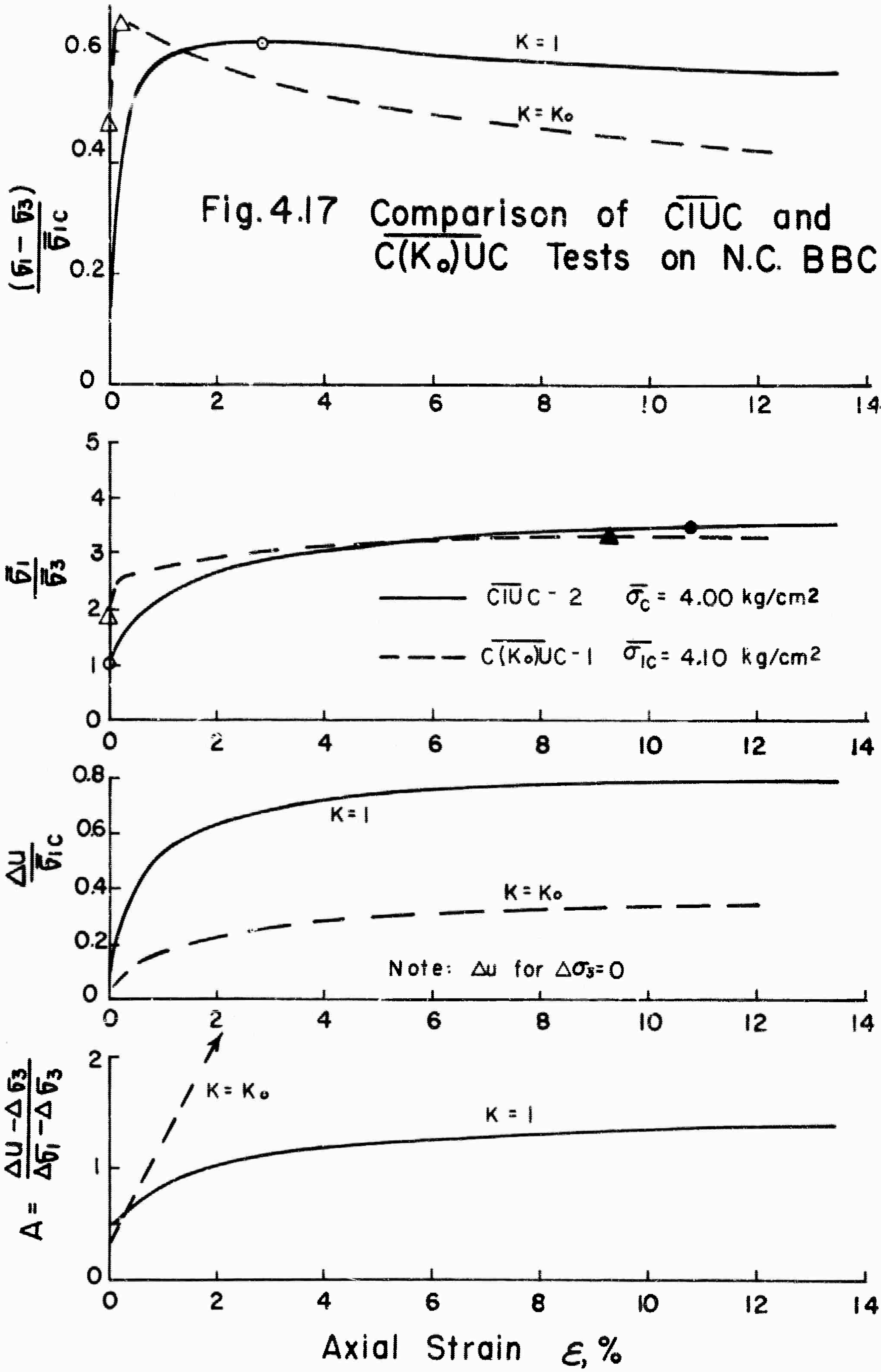


Fig. 4.17 Comparison of $\overline{\text{CIUC}}$ and $\overline{\text{C(K}_o\text{)UC}}$ Tests on N.C. BBC

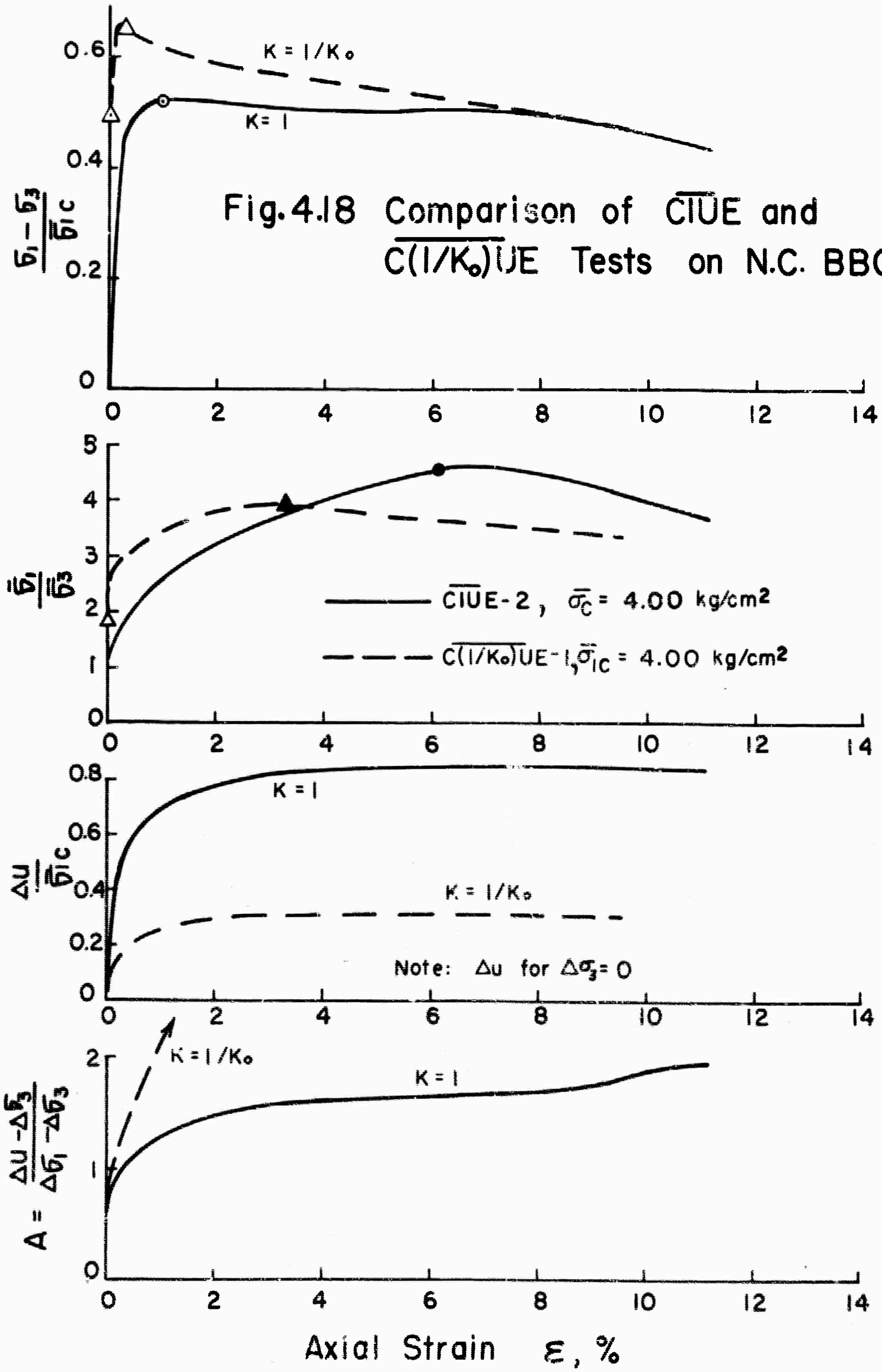


Fig. 4.18 Comparison of $\overline{\text{CIUE}}$ and $\overline{\text{C(1/K}_o\text{)UE}}$ Tests on N.C. BBC

Fig. 4.19 Effective Stress Paths from $\overline{CIU}$ and $\overline{C(K_o)U}$ Compression Tests on N.C. BBC

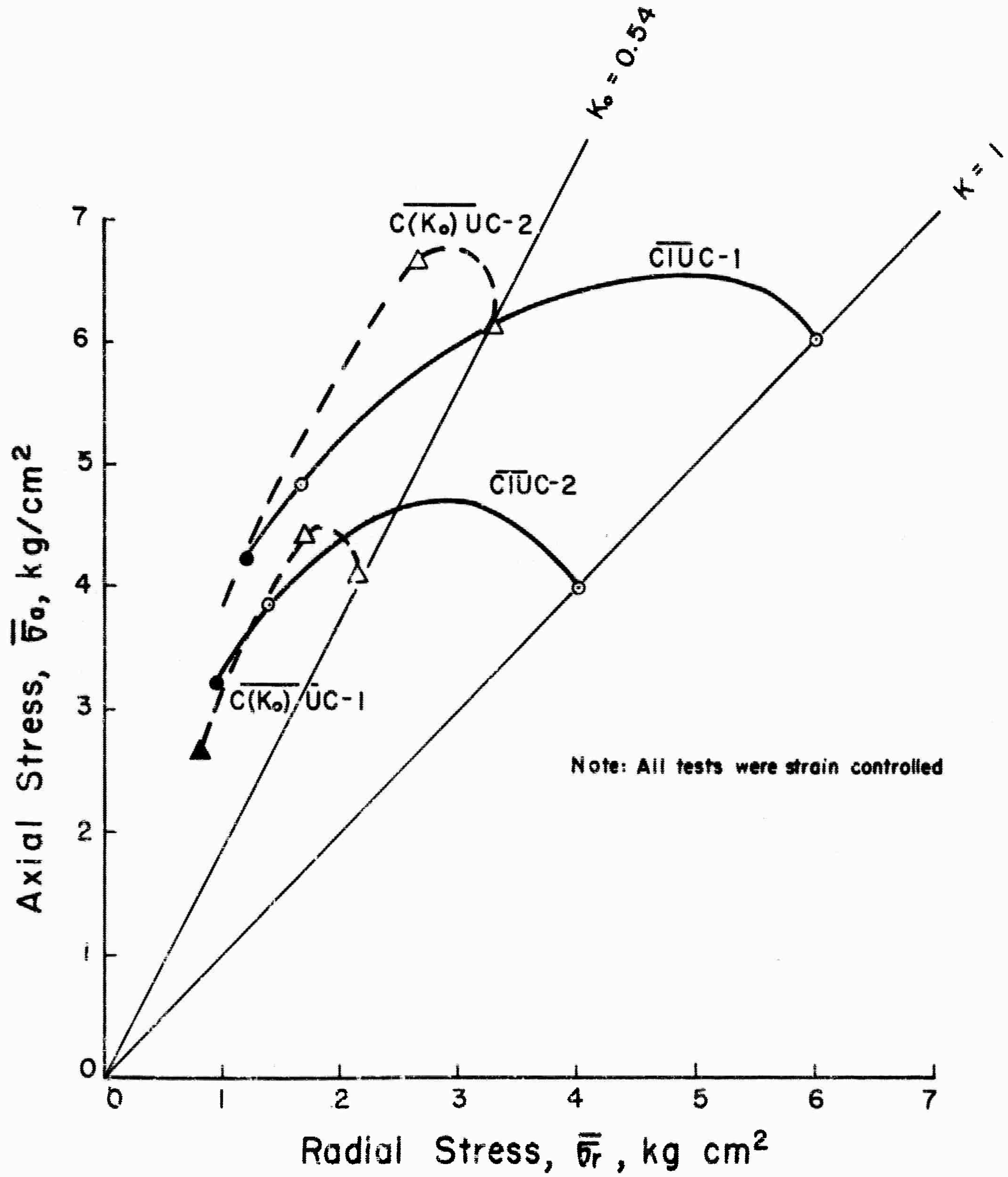
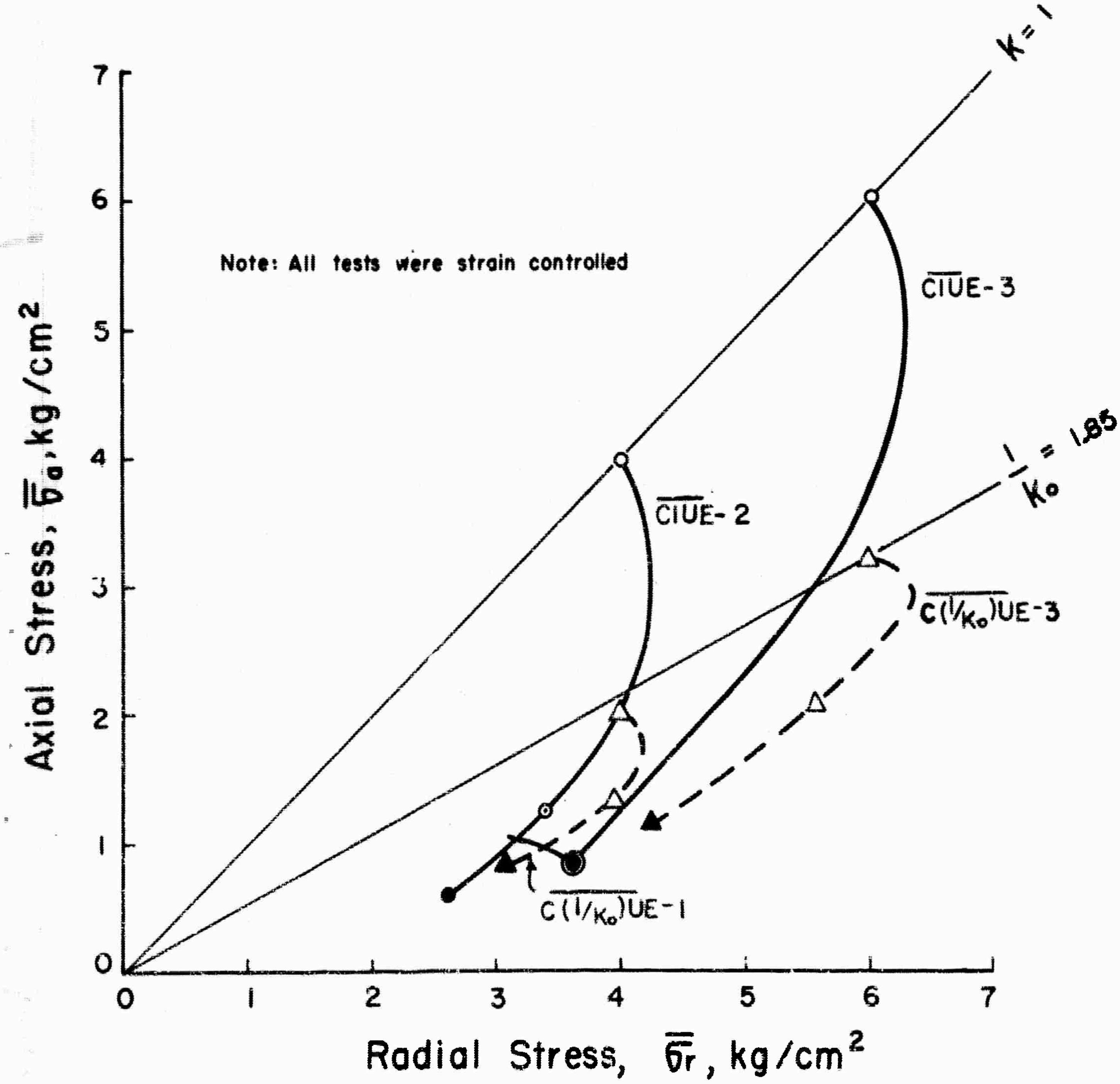


Fig. 4.20 Effective Stress Paths from $\overline{CIU}$ and $\overline{C(1/K_o)U}$ Extension Tests on N.C. BBC



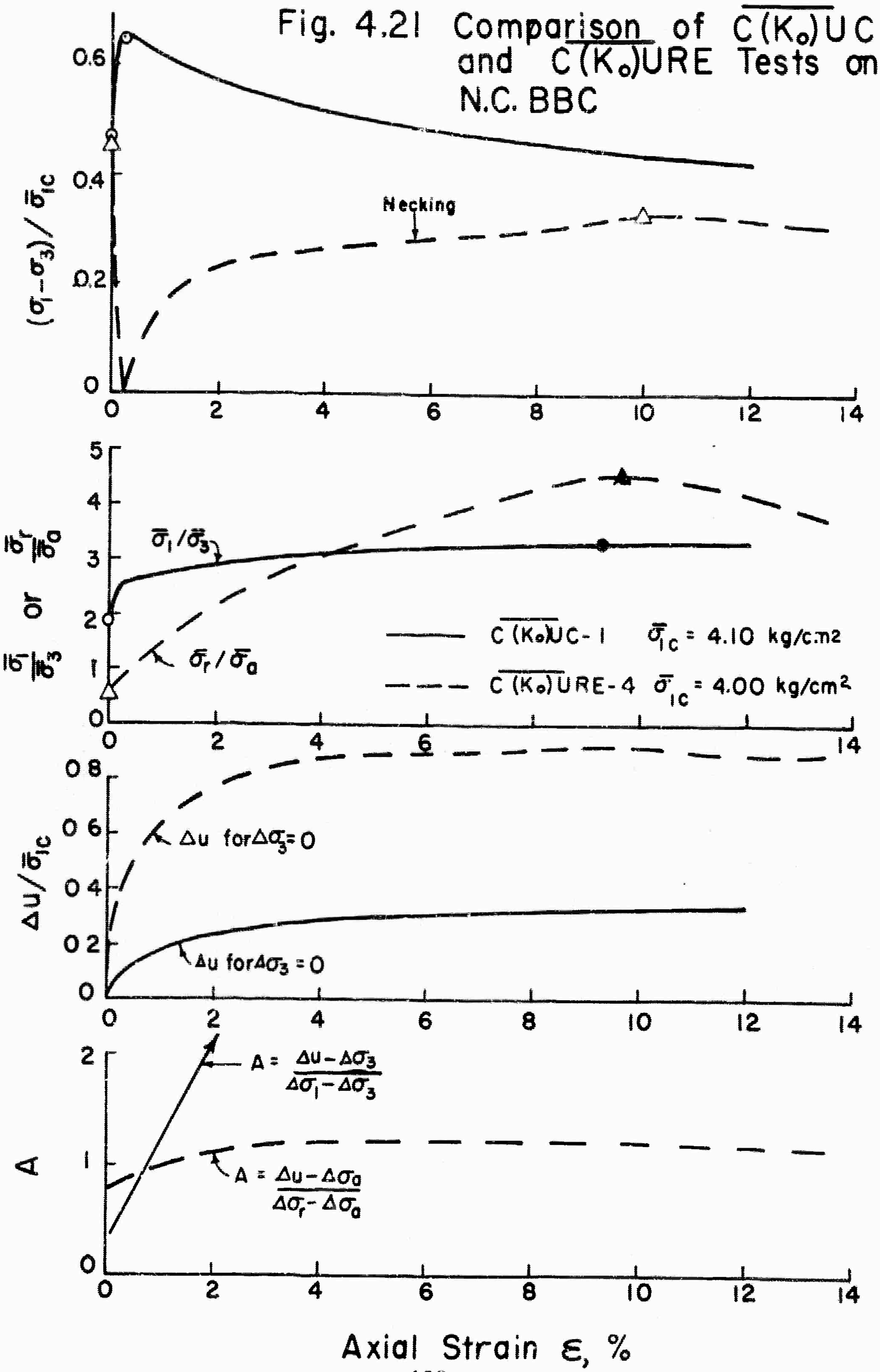


Fig. 4.21 Comparison of $\overline{C(K_o)UC}$ and $\overline{C(K_o)URE}$ Tests on N.C. BBC

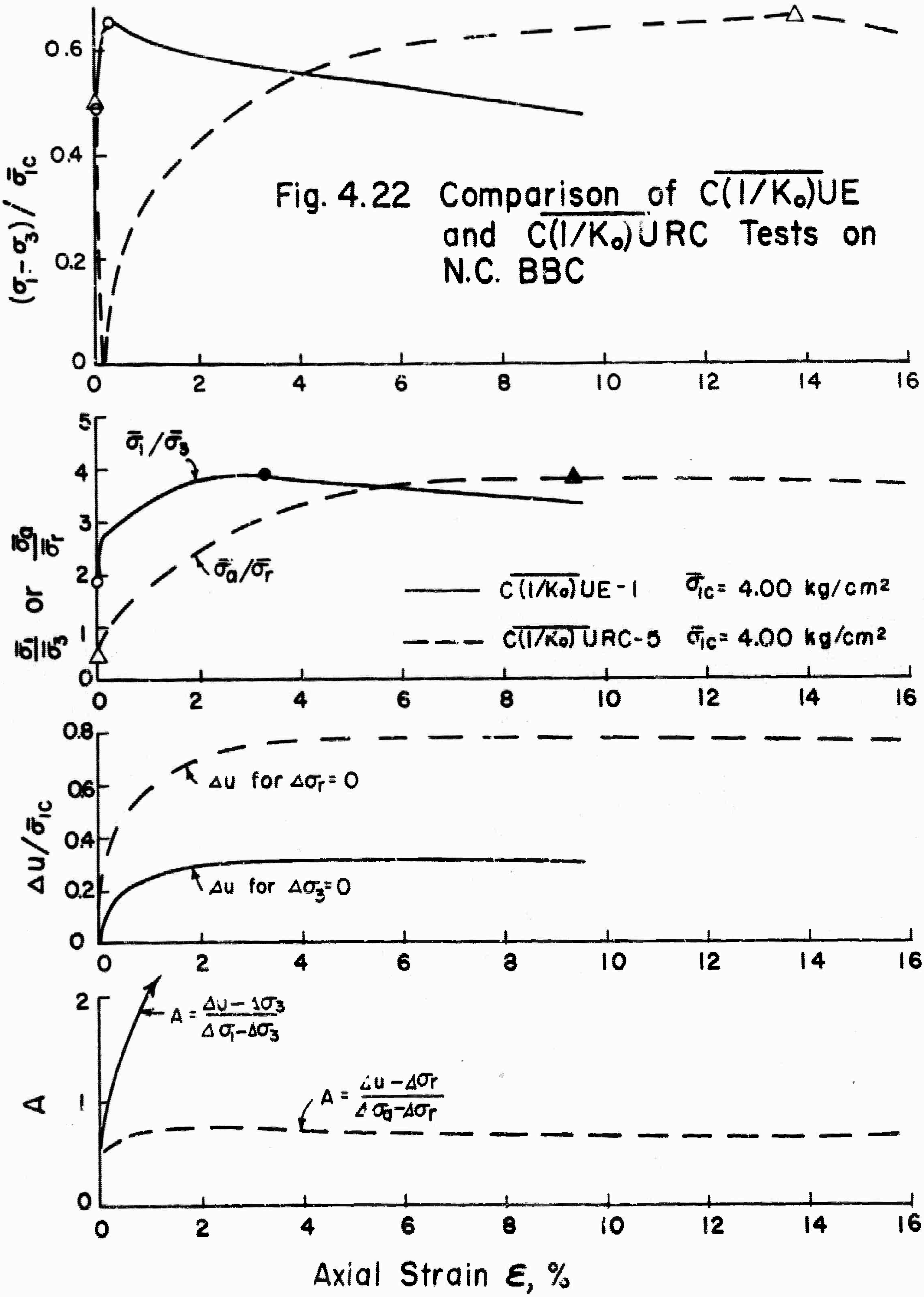


Fig. 4.22 Comparison of $C(\overline{1/K_o})UE$ and $C(\overline{1/K_o})URC$ Tests on N.C. BBC
$(\sigma_1 - \sigma_3)/\bar{\sigma}_{1c}$
$\bar{\sigma}_1/\bar{\sigma}_3$
$\bar{\sigma}_a/\bar{\sigma}_r$
$\bar{\sigma}_1/\bar{\sigma}_3$ or $\bar{\sigma}_a/\bar{\sigma}_r$
$C(\overline{1/K_o})UE-1$ $\bar{\sigma}_{1c} = 4.00$ kg/cm²
$C(\overline{1/K_o})URC-5$ $\bar{\sigma}_{1c} = 4.00$ kg/cm²
$\Delta u/\bar{\sigma}_{1c}$
Δu for $\Delta\sigma_r = 0$
Δu for $\Delta\sigma_3 = 0$
A
$A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}$
$A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$
Axial Strain ε, %

Fig. 4.23 Effect of Stress Path on Applied Stress vs Strain for $\overline{CU}$ Tests on N.C. BBC

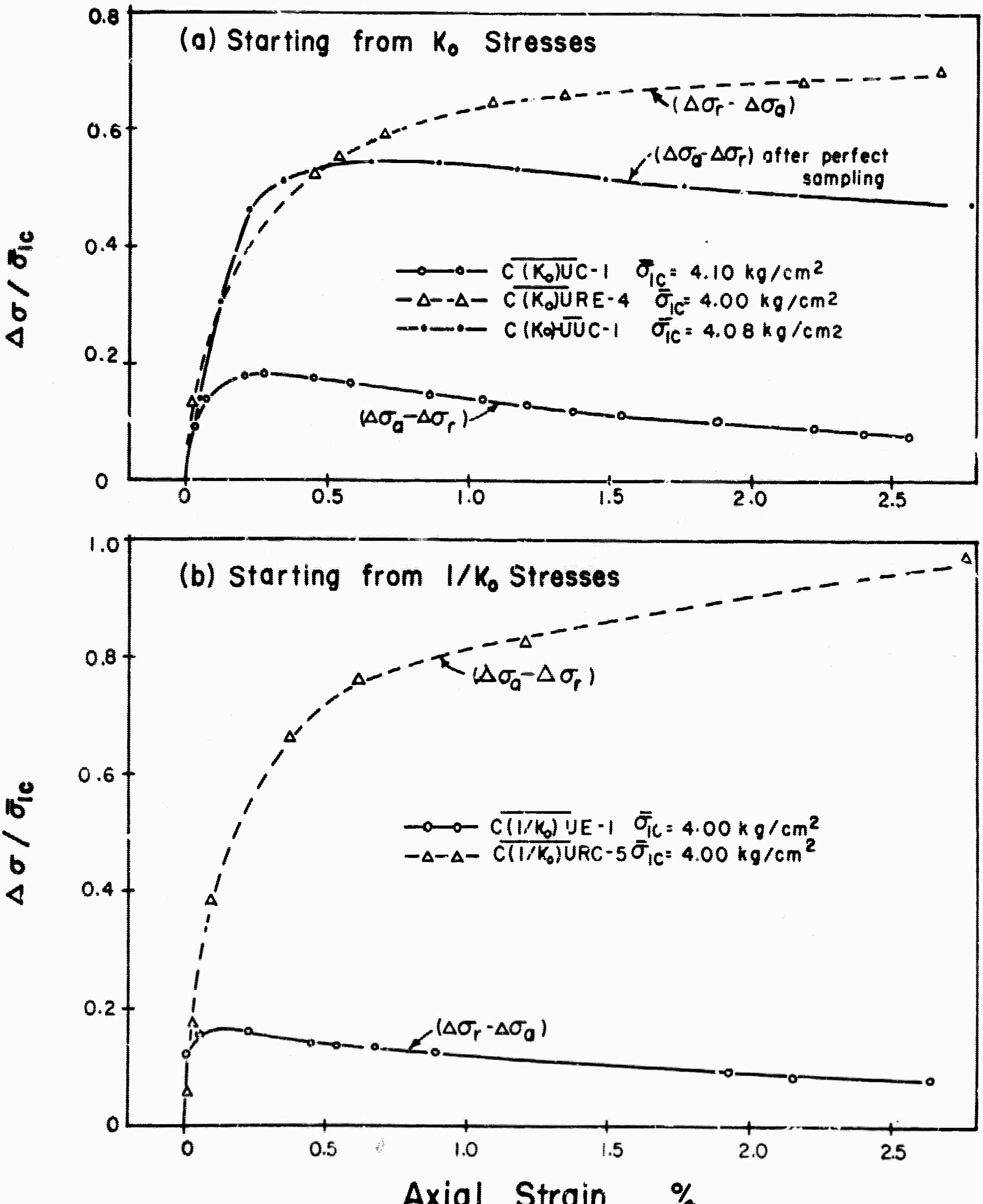
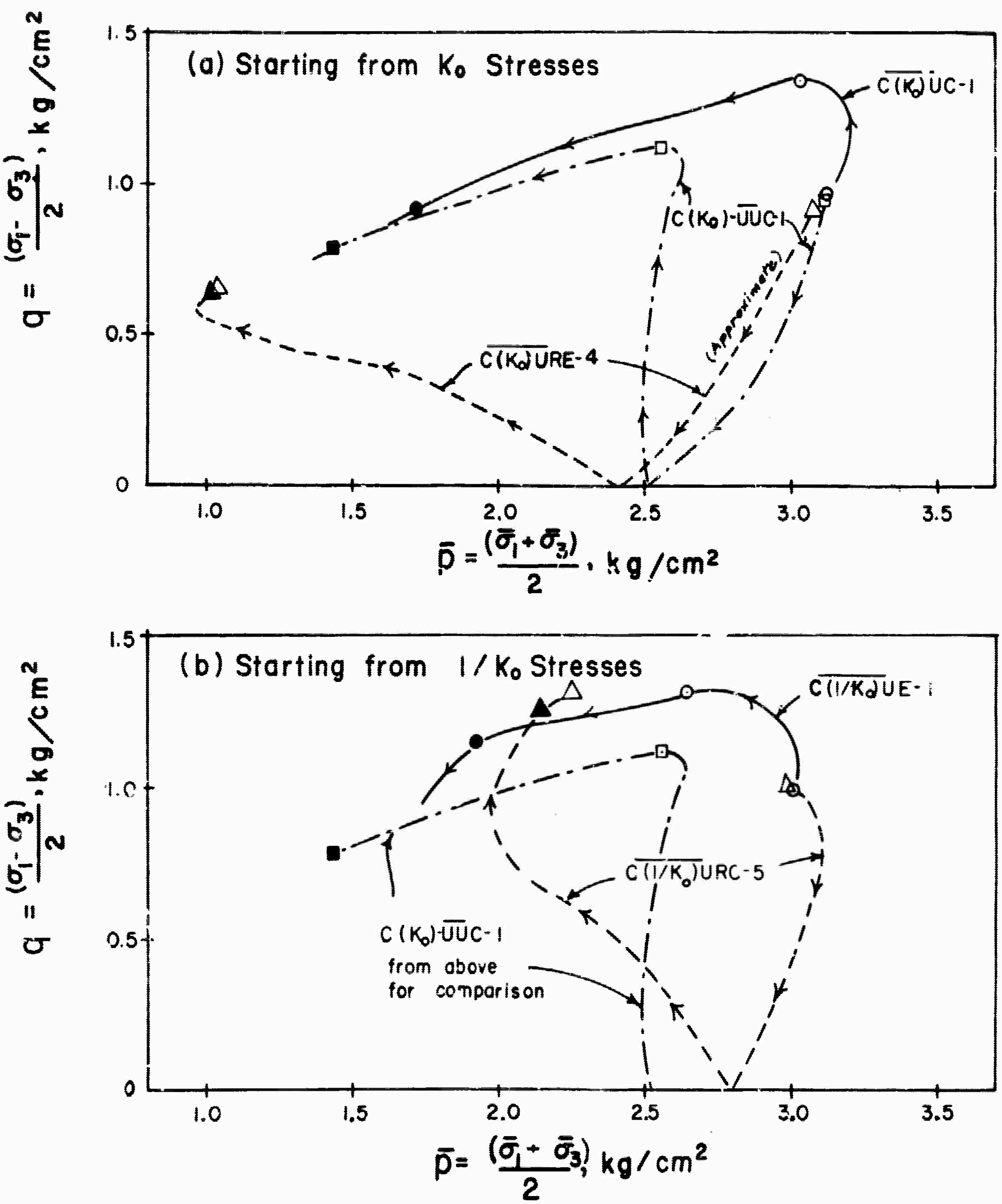


Fig. 4.24 Effective Stress Paths for $\overline{CU}$ Tests on N.C. BBC with and without Rotation of Principal Planes



CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 STATEMENT OF THE PROBLEM

Stability and deformation problems involving the undrained shear of deposits of saturated clay require a determination of the following soil properties:

s_u = in situ undrained shear strength;

$\bar{c}$, $\bar{\Phi}$ = effective stress parameters defining the Mohr-Coulomb failure envelope;

A = Skempton's pore pressure parameter;

E = Stress-strain modulus (or a stress-strain curve).

These soil properties are determined from field tests and/or laboratory shear tests run on "undisturbed" samples. Two basic requirements must be met in order to obtain measured properties which accurately duplicate the in situ behavior. These are:

1. Performance of tests on samples which have the same properties as the in situ clay. A field test such as the field vane meets this requirement (assuming negligible disturbance during insertion of the vane), but laboratory tests cannot because sample disturbance will cause a change in effective stress and/or water content prior to shear. For example, an unconfined compression test might have the in situ water content but it cannot duplicate the in situ preshear stresses. On the other hand, a triaxial CU test can duplicate the in situ

2. Perfect sampling

3. The intermediate principal stress

4. Rotation of principal planes during shear.

Some of the more important results are summarized below.

5.2.1 Anisotropic Consolidation

K_o consolidation, relative to isotropic consolidation (K = 1), of normally consolidated clays of moderate to high sensitivity will generally cause the following effects for undrained shear in triaxial compression:

1. Little change (±10 - 15%) in $s_u/\bar{\sigma}_{1c}$;
2. A decrease in A_f and $\bar{\phi}$ at failure,* although $\bar{\phi}$ at maximum obliquity is practically unchanged;
3. A large decrease in ϵ_f.

The assumption that effective stress paths from $\overline{\text{CIU}}$ tests represent an unique relationship among shear stress, effective stress, and water content is not valid and will usually lead to underestimates of $s_u/\bar{\sigma}_{1c}$ for anisotropically consolidated samples

Little data are available on the effect of anisotropic consolidation on the strength behavior of overconsolidated clays. Assuming the validity of Principle I (p. 7 of Part I which states that there is an unique relationship among water content, shear stress and effective stress), one would predict the following effects of anisotropic consolidation:**

1. For K < 1, $s_u/\bar{\sigma}_{vc}$ would be decreased;
2. For K > 1. $s_u/\bar{\sigma}_{vc}$ would usually be increased

---

* Failure is defined as $(\sigma_1 - \sigma_3)_{max}$.

** See Fig. III-9 of Part I for the effective stress paths from which these trends are obtained.

2. Perfect sampling

3. The intermediate principal stress

4. Rotation of principal planes during shear.

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** See Fig. III-9 of Part I for the effective stress paths from which these trends are obtained.

relative to isotropic consolidation, where $\bar{\sigma}_{vc}$ = vertical consolidation stress.

5.2.2 Perfect Sampling

Perfect sampling denotes an undrained release of K_o shear stress to attain an isotropic state of stress. The effective stress after perfect sampling, $\bar{\sigma}_{ps}$, is given by:

$$\bar{\sigma}_{ps} = \bar{\sigma}_{vc}\,[K_o + A_u(1 - K_o)]$$

where

$\bar{\sigma}_{vc}$ = vertical consolidation stress

A_u = pore pressure parameter for unloading
= $(\Delta u - \Delta\sigma_h)/(\Delta\sigma_v - \Delta\sigma_h)$

$\Delta\sigma_v$ = change in vertical total stress

$\Delta\sigma_h$ = change in horizontal total stress

$(\Delta\sigma_v - \Delta\sigma_h)$ = $-\bar{\sigma}_{vc}(1 - K_o)$ for perfect sampling.

For normally consolidated clays:

A_u = $+0.15 \pm 0.15$

K_o = 0.6 ± 0.2

and

$\bar{\sigma}_{ps}/\bar{\sigma}_{vc}$ = 0.66 (0.40 - 0.86).

In other words, the effective stress after perfect sampling will generally be somewhat larger than the in situ horizontal consolidation stress.

As clays become overconsolidated, the value of K_o increases, A_u probably increases, and $\bar{\sigma}_{ps}/\bar{\sigma}_{vc}$ increases. For a heavily overconsolidated clay, perfect sampling will yield an effective stress exceeding the vertical consolidation stress.

The undrained compressive strength of perfect samples of normally consolidated clays is 10 ± 5 per cent less than the in situ strength in compression, while $\bar{\Phi}_u$ may be slightly increased and A_f will probably be considerably decreased.

5.2.3 Intermediate Principal Stress

Triaxial extension, relative to triaxial compression, generally:

1. Decreases s_u by 20 ± 10% because of large increases in excess pore pressures with the higher value of σ_2;
2. May increase $\bar{\Phi}$ at both $(\sigma_1 - \sigma_3)_{max.}$ and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max.}$ for normally consolidated clays (the data are not conclusive; some researchers report no influence, others report increases of several degrees);
3. Increases the effective stress envelope of over-consolidated clays;
4. Increases the stress-strain modulus, at least at low stress levels.

For plane strain, where $\bar{\sigma}_{2f}$ probably equals $(0.4 \pm 0.1) \times (\bar{\sigma}_1 + \bar{\sigma}_3)_f$, little data are available. It appears, however, that plane strain has a much smaller effect on strength behavior than triaxial extension. As a first approximation, one might assume that parameters for plane strain fall midway between those for triaxial compression and triaxial extension.

5.2.4 Rotation of Principal Planes

Rotation of principal planes refers to a change in the direction of the principal planes during shear. There are essentially no data which conclusively show the influence of rotation of principal planes at a constant value of the intermediate principal stress. However, there is indirect evidence which indicates that rotation per se will produce increased excess pore pressures during undrained shear of normally consolidated clays, with a resultant decrease in undrained shear strength. The effect may be very important.

5.2.5 Non-Uniqueness of Undrained Strength

Engineering practice commonly assumes that the undrained shear strength of a clay is uniquely related to consolidation stress and/or water content at failure. Yet, the preceding sections show that this assumption is not valid and, moreover, that such an assumption may err on the unsafe side. This fact is illustrated by the results presented in Table 5.1 from shear tests on three normally consolidated clays. Values of undrained shear strength are shown for four types of tests:

1. $\overline{CIUC}$, a common triaxial compression test on an isotropically consolidated sample;
2. $\overline{C(K_o)UC}$, a triaxial compression test on a K_o consolidated sample, which represents shear under the center line of a circular footing (no rotation of principal planes);
3. $\overline{C(K_o)URE}$, a triaxial extension test on a K_o consolidated sample, which represents shear under the center line of a circular excavation (rotation of principal planes);
4. Simple shear, a "high class" constant volume direct shear test on a K_o consolidated sample, which represents shear along a horizontal failure plane.

It is seen that the values of $s_u/\bar{\sigma}_{vc}$ (undrained strength divided by vertical consolidation stress) for samples with the same consolidation stress and/or water content are anything but constant. Undrained shear for an excavation and for a horizontal failure plane produce strengths only one-half to three-quarters of the strength for shear under a footing. These are certainly large differences which can have practical significance. There would, of course, be commensurate changes in other parameters, such as the friction angle, the pore pressure parameter, and the stress-strain modulus.

The most important stress system variables controlling undrained strength behavior are probably the value of K at consolidation, the relative value of σ_2 at failure, and the direction of σ_1 at failure relative to its direction after consolidation (i.e., rotation of principal planes). These variables are likely to be especially significant with normally consolidated clays of moderate to high sensitivity. Overconsolidated clays of low sensitivity sheared from a K_o condition which is close to unity should be relatively little affected by variations in the stress system during shear.

Furthermore, one cannot account for the influence of stress systems on undrained shear strength by simply considering variations in the consolidation stress on the failure plane, $\bar{\sigma}_{fc}$. The data in Table 5.1 show that the ratio $s_u/\bar{\sigma}_{fc}$ is even more variable than the ratio $s_u/\bar{\sigma}_{vc}$. Rather, one should attempt to perform shear tests which duplicate the in situ stress system, both prior to and during shear.

5.3 VALIDITY OF THE $\phi = 0$ STABILITY ANALYSIS

5.3.1 Current Practice

The $\phi = 0$ stability analysis is employed to compute the factor of safety of footings, excavations, embankments, etc., with respect to an undrained shear failure. The analysis requires a determination of the in situ undrained shear strength, s_u. This strength is commonly evaluated from the results of field vane tests, triaxial unconfined or UU compression tests, and/or triaxial CU compression tests. Although these three methods often yield quite different results (see Section 1.1), Bishop and Bjerrum (1960) report excellent agreement between computed and observed failures, wherein s_u was obtained from field vane and/or triaxial U (or UU) compression tests, for a great variety of footings, embankments, cuttings, and strutted excavations in K_o consolidated clays.* On the other hand, for natural slopes (non-K_o consolidation), the $\phi = 0$ analysis usually yields too high a factor of safety with overconsolidated clays and too low a value with normally consolidated clays. The questions then arise:

1. In light of the influence of stress system on undrained strength, how can the $\phi = 0$ analysis, which assumes an unique in situ strength which can be obtained from field vane of UU compression tests, be so successful in K_o consolidated clays?
2. Why should the method work with K_o consolidated clays, but not with non-K_o consolidated clays such as encountered in natural slopes or in clay strata consolidated under an overlying embankment or footing?

It is hypothesized that the reported successful use of the $\phi = 0$ analysis with K_o consolidated clays may often depend upon compensating errors, at least with normally consolidated clays. For example, a decrease in the value of the in situ s_u (relative to s_u for shear in compression) caused by different total stress paths and a decrease in the s_u of unconfined compression tests caused by disturbance during sampling (Ladd and Lambe, 1963) are possible compensating errors. On the other hand, the errors apparently do not balance each other with non-K_o consolidated clays.

An example of the above hypothesis is presented in the next section.

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* Excavations in stiff-fissured clays are an exception. Such clays will be excluded from further discussion as they are a special case.

5.3.2 Hypothetical Field Cases

The following hypothetical data illustrate the variation in undrained strengths that might typically result from field vane tests and laboratory triaxial tests on "undisturbed" tube samples of a normally consolidated clay. These data are then used to compute the stability of a vertical cut, a retaining wall with a passive thrust, and a strip footing, all for undrained shear of the normally consolidated clay. Finally, these computed values are compared with in situ values for an undrained shear failure wherein the appropriate value of the in situ s_u was chosen* on the basis of data trends presented (or reviewed) in this report.

Table 5.2 presents the values of strength determined from the field vane test and from the various laboratory tests and compares computed factors of safety for the three field cases with actual values.

Regarding the methods of determining s_u from field and laboratory tests:

1. The ratio $s_u/\bar{\sigma}_{vc} = 0.28$ represents the strength in triaxial compression of a "perfect sample" (no disturbance other than release of the in situ shear stress). It might be obtained from a truely undisturbed block sample or by adjusting measured UU or CIU triaxial compression test data for the effects of sample disturbance (such as by the methods suggested by Ladd and Lambe, 1963).

2. The laboratory unconfined tests yields a lower

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* The values chosen cannot be supported by actual test data because none exist. They are the authors' estimate of what are thought to be reasonable values using data on Boston blue clay as a starting point. In any case, they are only used to illustrate the meaning of an hypothesis.

strength ($s_u/\bar{\sigma}_{vc} = 0.21$) because sample disturbance decreases the preshear effective stress to a value much below that corresponding to the perfect sample, $\bar{\sigma}_{ps}$.

3. The laboratory CIU triaxial compression test on a sample isotropically consolidated to the in situ vertical effective stress, $\bar{\sigma}_{vo}$, yields a strength higher than that of the perfect sample because $\bar{\sigma}_{vo}$ is greater than $\bar{\sigma}_{ps}$ and because of the appreciable volume decrease during reconsolidation as a result of sample disturbance.

4. The field vane yields a strength falling between those from the unconfined and CIU tests, and which agrees with the compressive strength of a perfect sample. There are cases wherein the field vane gives strengths below those from unconfined tests on good samples (for example, the Leda clay and the Manglerud clay from Oslo) and other cases wherein the field vane strengths exceeded those from CIU tests consolidated to $\bar{\sigma}_{vo}$. There is certainly no consistent, reliable relationship between field vane and laboratory unconfined tests on normally consolidated clays. Whenever the in situ K is not close to unity, it is very difficult to interpret the significance of field vane tests because of the very complicated stress system (see Fig. 2.22).

In summary, the results of field vane, and laboratory unconfined and CIU tests will usually yield quite different results, all of which are likely to be different from the triaxial compressive strength of a perfect sample.

Now, how do the above strengths compare with in situ strengths for the three cases in Table 5.2? First of all, it is apparent that the in situ strength depends on the type of stability problem under consideration. For the vertical cut, there is plane strain shear with no rotation of principal planes. The resulting value of $s_u/\bar{\sigma}_{vo}$ equals 0.295, which is somewhat below that for in situ triaxial compression ($s_u/\bar{\sigma}_{vo}$ = 0.33).* Shear is also in plane strain for the passive thrust on the retaining wall, but here the principal planes rotate by 90^o (σ_1 starts in the vertical direction and ends up in the horizontal direction). The resulting value of $s_u/\bar{\sigma}_{vo}$ equals 0.20, a considerable reduction from the previous case.** The strip footing involves three states of shear: plane strain with no rotation of principal planes in the active wedge; plane strain with an approximately horizontal failure plane and rotation of principal planes; and plane strain with rotation in the passive wedge. Each section has a different value of strength; the average $s_u/\bar{\sigma}_{vo}$ equals 0.21.

A comparison of computed and actual factors of safety (Table 5.2) shows "excellent agreement" in some instances, but it is obvious that such agreement is largely fortituous. The laboratory unconfined strength on a disturbed sample predicted the in situ strength for the retaining wall and footing, but not for the vertical cut. The compressive strength of a perfect sample worked well for the cut, but overestimated strengths for the other cases, as did the field vane (again it is emphasized that the field vane does not yield consistent strengths). The CIU test consolidated to the in situ vertical stress always yielded results which were too high (if a $s_u/\bar{\sigma}_c$ ratio had been determined at $\bar{\sigma}_c \geq$ 2 to 4 times $\bar{\sigma}_{vo}$, the resulting strengths would have been in close agreement to the compressive strength of the perfect sample).

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* See Table 4.3 for the results of $\overline{C(K_o)UC}$ tests on BBC.

** The strength of an element below the center line of a vertical excavation would equal 0.155 (see Table 4.3 for a $\overline{C(K_o)URE}$ test).

In summary, the foregoing hypothetical numbers (but derived from data on the Boston blue clay) illustrate four important points:

1. The in situ undrained strength depends upon the mode of failure;
2. Methods commonly used to predict in situ strengths for a $\phi = 0$ analysis are liable to yield very different results;
3. The success of a $\phi = 0$ analysis may depend to a large degree on compensating errors, yet one can not predict with certainty when the errors will balance out and when they will not;
4. Interpretation of the field vane is very difficult, if not currently impossible, when the in situ K is not close to unity.

5.4 CONCLUSION

The prediction of the in situ parameters of a clay deposit for undrained shear from the results of laboratory tests requires:

1. Testing of samples which have, or closely duplicate, the properties of the in situ material;
2. Testing of samples in a manner so as to measure the correct properties.

The sampling process precludes exact duplication in the laboratory of the in situ water content and effective stress, even for "perfect sampling." Disturbance during tube sampling generally causes laboratory samples to have a much lower effective stress than for perfect sampling. In many clays it may be possible to overcome the most serious effects of sample disturbance by reconsolidation in the laboratory.

One of the most important factors to consider in attempting to measure the correct shear parameters is the stress system applied prior to and during shear. This requires duplication of the in situ K at consolidation, the relative value of the intermediate principal stress during shear, and the direction of the principal stresses, especially regarding rotation during shear.

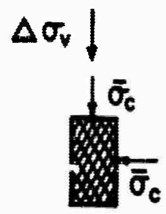
It is emphasized that the in situ undrained shear strength, s_u, depends on the mode of failure (i.e., in situ stress system). It is not a constant as commonly assumed in practice. For a normally consolidated clay, the in situ strength for a strutted excavation or for an embankment may be 20 to 40% lower than the in situ strength for a vertical cut or for a retaining wall with an active pressure. There is no reason why a compression test on a truely undisturbed sample should yield the in situ strength of a clay.

Consequently, one cannot use the results of a single type of field or laboratory shear test with equal success for all types of $\phi = 0$ stability analysis. The utilization of unconfined compression tests and field vane tests depends upon a cancellation of errors. Therefore, these tests alone should not be relied upon for $\phi = 0$ stability analyses involving important structures unless considerable experience with the particular clay in question has already developed a reliable set of empirical guidelines.

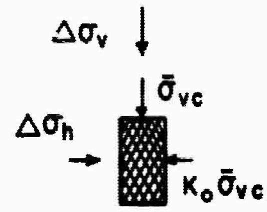
Table 5.1 Undrained Strength of Normally Consolidated Clay vs Type of Shear

A. Types of Shear Tests

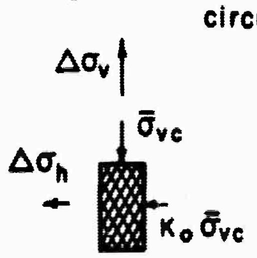
(1) $\overline{\text{CIUC}}$ (Common practice)



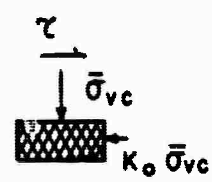
(2) $\overline{\text{C(K}_o\text{)UC}}$ (Below centerline of circular footing)



(3) $\overline{\text{C(K}_o\text{)URE}}$ (Below centerline of circular excavation)



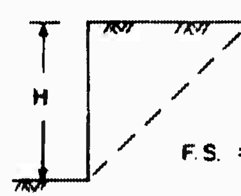
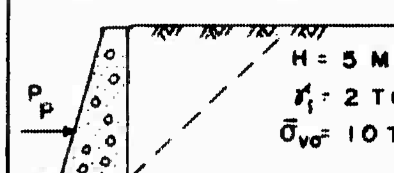
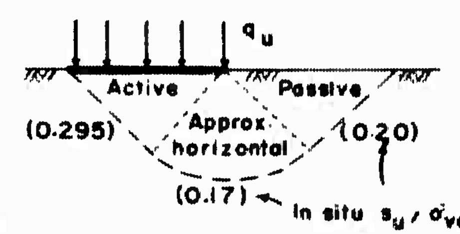
(4) Simple Shear (Horizontal failure plane)



B. Undrained Strengths

Clay	Type of Test	$s_u/\bar{\sigma}_{vc}$	$s_u/\bar{\sigma}_{fc}$
Boston Blue Clay (Remolded)	$\overline{\text{CIUC}}$	0.285	0.285
	$\overline{\text{C(K}_o\text{)UC}}$	0.33	0.50
	$\overline{\text{C(K}_o\text{)URE}}$	0.155	0.17
Kawasaki Clay II (Undisturbed)	$\overline{\text{CIUC}}$	0.475	0.475
	$\overline{\text{C(K}_o\text{)UC}}$	0.445	0.745
	$\overline{\text{C(K}_o\text{)URE}}$	0.225	0.24
Manglerud Clay (Undisturbed)	$\overline{\text{CIUC}}$	0.29	0.29
	$\overline{\text{C(K}_o\text{)UC}}$	0.28	0.435
	Simple Shear	0.21	0.21

Table 5.2 Comparison of Computed and Actual Factors of Safety for Undrained Shear of a Hypothetical Normally Consolidated Clay

Method of Determining s_u	$s_u/\bar{\sigma}_{vo}$	Vertical Cut	Retaining Wall — Passive Thrust	Strip Footing
		$\frac{\text{F.S. computed}}{\text{F.S. actual}}$	$\frac{P_p \text{ computed}}{P_p \text{ actual}}$	$\frac{q_u \text{ computed}}{q_u \text{ actual}}$
Lab Unconfined Compression	0.21	0.71	1.02 (P_p computed = 46)	1.00
Field Vane	0.275 ($\tau/\bar{\sigma}_{vo}$)	0.93	1.17 (P_p computed = 52.5)	1.31
Lab CIU Compression, $\bar{\sigma}_c = \bar{\sigma}_{vc}$	0.38	1.29	1.40 (P_p computed = 63)	1.81
Lab UU Compression, "perfect sample"	0.28	0.95	1.18 (P_p computed = 53)	1.33
		Plane strain, no rotation of P.P.  H $\text{F.S.} = \frac{4 s_u}{\gamma_t H}$	Plane strain, rotation of P.P.  P_p H = 5 M γ_t = 2 TCM $\bar{\sigma}_{vo}$ = 10 TSM $P_p = \frac{1}{2}\gamma_t H^2 + 2 s_u H$ $= 25 + 100 (s_u/\bar{\sigma}_{vo})$	 q_u Active Passive Approx. horizontal (0.295) (0.20) (0.17) In situ $s_u/\bar{\sigma}_{vo}$ $q_u = 5.7\, s_u$
In Situ $s_u/\bar{\sigma}_{vo}$		0.295	0.20	0.21 (average)

CHAPTER 6

RECOMMENDATIONS

Further research on the influence of stress system on the strength and deformation characteristics of saturated clays is recommended in the following areas:

1. Perform triaxial tests, of the types reported herein on normally consolidated BBC, on several different types of clay (normally consolidated and overconsolidated and of varying sensitivity) in order to obtain a better perspective of the general importance of stress system.

2. Employ shear test devices which better duplicate common in situ stress systems. A true triaxial apparatus in which all three principal stresses can be varied would be ideal, but it would require considerable development costs with no guaranty of success. A hollow cylinder shear device poses serious problems regarding non-uniformity of stresses and methods of determining stresses and strains. It would appear that the most feasible approach should employ existing equipment such as the N.G.I. constant volume direct shear device (simple shear machine) and a variation of Bishop's plane strain device. These equipment could yield extremely useful data in arriving at an overall picture of the effects of stress system.

3. Perform drained shear tests of the above types

in order to obtain a better understanding of the significance of Lambe's stress path method for computation of settlements.

4. Compare the results of laboratory determined parameters with those backfigured from field and/or model tests. Verification of the hypothesis and approaches proposed in this report depends ultimately upon what is observed in the field.

CHAPTER 7

REFERENCES

Note: ICSMFE = International Conference on Soil Mechanics and Foundation Engineering

JSMFD = Journal of Soil Mechanics and Foundations Division

RCSSCS = Research Conference on the Shear Strength of Cohesive Soils, Boulder, Colorado.

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APPENDIX A

LIST OF NOTATIONS

Note: Suffix f indicates a failure condition.
Prefix Δ indicates a change.
A bar over a stress indicates an effective stress.

1. Stresses

u	Pore water pressure
σ	Total normal stress
$\bar{\sigma}$	Effective normal stress
σ_c	Chamber (cell) pressure
$\bar{\sigma}_c$	Consolidation pressure
σ_a, $\bar{\sigma}_a$	Axial stress
σ_r, $\bar{\sigma}_r$	Radial stress
σ_h, $\bar{\sigma}_h$	Horizontal stress
σ_v, $\bar{\sigma}_v$	Vertical stress
σ_1, $\bar{\sigma}_1$	Major principal stress
σ_2, $\bar{\sigma}_2$	Intermediate principal stress
σ_3, $\bar{\sigma}_3$	Minor principal stress
$\bar{\sigma}_{ac}$, $\bar{\sigma}_{rc}$	$\bar{\sigma}_a$, $\bar{\sigma}_r$ at consolidation
$\bar{\sigma}_{hc}$, $\bar{\sigma}_{vc}$	$\bar{\sigma}_h$, $\bar{\sigma}_v$ at consolidation
$\bar{\sigma}_{1c}$, $\bar{\sigma}_{2c}$, $\bar{\sigma}_{3c}$	$\bar{\sigma}_1$, $\bar{\sigma}_2$, $\bar{\sigma}_3$ at consolidation
$\bar{\sigma}_{fc}$	Consolidation stress on failure plane
$\bar{\sigma}_{ff}$	Effective stress on failure plane at failure

Stresses

$\bar{\sigma}_{ho}$, $\bar{\sigma}_{vo}$	In situ horizontal, vertical effective stresses
$\bar{\sigma}_{ps}$	Effective stress after perfect sampling from a K_o condition
$\bar{\sigma}_s$	Effective stress of a sample after actual sampling
τ	Shear stress
τ_{ff}	Shear stress on failure plane at failure
q	$(\sigma_1 - \sigma_3)/2$
q_c	q at consolidation
q_f	q at failure
p	$(\sigma_1 + \sigma_3)/2$
$\bar{p}$	$(\bar{\sigma}_1 + \bar{\sigma}_3)/2$
$\bar{p}_c$	$\bar{p}$ at consolidation
$\bar{p}_f$	$\bar{p}$ at failure
s_u	q_f for undrained shear
s_d	q_f for drained shear

2. Stress Ratios

A	Skempton's A parameter $= (\Delta u - \Delta\sigma_3)/(\Delta\sigma_1 - \Delta\sigma_3)$
A_f	A parameter at failure, i.e., $(\sigma_1 - \sigma_3)_{max.}$
A_u	A parameter for unloading $= (\Delta u - \Delta\sigma_h)/(\Delta\sigma_v - \Delta\sigma_h)$
B	Skempton's B parameter $= \Delta u/\Delta\sigma_c$
K	$\bar{\sigma}_r/\bar{\sigma}_a$ or $\bar{\sigma}_h/\bar{\sigma}_v$ or $\bar{\sigma}_{hc}/\bar{\sigma}_{vc}$
K_c	$\bar{\sigma}_{hc}/\bar{\sigma}_{vc}$
K_o	K_c for no lateral strain = coefficient of earth pressure at rest

Stress Ratios

K_f	K at failure
OCR	Overconsolidation Ratio = $\bar{\sigma}_{cm}/\bar{\sigma}_c$ or $\bar{\sigma}_{vm}/\bar{\sigma}_{vc}$

3. Strength and Stress-Strain Parameters

$\bar{c}$	Cohesion intercept of a τ_{ff} versus $\bar{\sigma}_{ff}$ envelope
$\bar{\phi}$	Friction angle of τ_{ff} versus $\bar{\sigma}_{ff}$ envelope
$\bar{\phi}_u$	$\bar{\phi}$ at $(\sigma_1 - \sigma_3)_{max.}$ for undrained shear
$\bar{a}$	Intercept of a q_f versus $\bar{p}_f$ envelope
$\bar{\alpha}$	Slope of a q_f versus $\bar{p}_f$ envelope
E	Stress-strain modulus = $\Delta(\sigma_1 - \sigma_3)/\epsilon$

4. Miscellaneous

e	Void ratio
G_s	Specific gravity of soil solids
S	Degree of saturation
w	Water content
w_i	Initial water content
w_f	Water content at failure
w_N	Natural water content
w_L	Liquid limit
w_P	Plastic limit
P.I. = I_p	Plasticity index
L.I. = I_L	Liquidity index
t	Time
t_c	Time of consolidation under the last increment

Miscellaneous

t_f	Time to failure
F.S.	Factor of safety
ϵ	Axial strain
$\epsilon_1, \epsilon_2, \epsilon_3$	Linear strains in direction of the three principal stresses
ϵ_a, ϵ_r	Linear strains in axial and radial directions

5. Types of Shear Tests

CD	Consolidated-Drained
CU	Consolidated-Undrained
$\overline{\text{CU}}$	CU test with pore pressure measurements
UU, $\overline{\text{UU}}$	Unconsolidated-Undrained

6. Types of Triaxial Tests

CIU, $\overline{\text{CIU}}$	CU, $\overline{\text{CU}}$ test with isotropic consolidation
CAU, $\overline{\text{CAU}}$	CU, $\overline{\text{CU}}$ test with anisotropic consolidation
$\overline{\text{CIU}}$C	Compression test on isotropically consolidated sample
$\overline{\text{CIU}}$E	Extension test on isotropically consolidated sample
$\overline{\text{C(K}_o\text{)U}}$C	Compression test on K_o consolidated sample
$\overline{\text{C(K}_o\text{)U}}$RE	Extension test on K_o consolidated sample
$\overline{\text{C(1/K}_o\text{)U}}$E	Extension test on $1/K_o$ consolidated sample
$\overline{\text{C(1/K}_o\text{)U}}$RC	Compression test on $1/K_o$ consolidated sample
C(K_o)-$\overline{\text{UU}}$C	Compression test on perfect sample after K_o consolidation

APPENDIX B

TABLES OF SOIL PROPERTIES AND STRESS-STRAIN DATA FROM $\overline{CU}$ TRIAXIAL TESTS ON NORMALLY CONSOLIDATED BOSTON BLUE CLAY PREPARED FROM A DILUTE SLURRY

Soil Properties

Test No. $\overline{CIUC}$-2   Tested by J.V.   7/1963 $\bar{\sigma}_c = 4.00$ kg/cm²

Batch No. S-5 $d\varepsilon/dt = 0.0006$ inch/min.   P.P.R. = 100 %

8 vertical paper drains   back pressure 3.00 kg/cm²

σ_1 and σ_3 varied $t_c = 4$ days

$\bar{\sigma}_c$ kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	80.80	8.00	10.10	31.4	94.4	.925	—	1.90
1.00	78.52	—	—	29.6	95.0	.87	2.6	1.93
2.00	76.41	—	—	28.1	95.4	.82	5.2	1.96
4.00	71.40	7.80	9.16	25.4	100	.704	11.5	2.05

circ. at end of test : not measured

Stress - Strain Data

Test No. $\overline{CIUC}$-2   Tested by J.V.   7/1963 $\bar{\sigma}_c = 4.00$ $w_i = 31.4\%$

$w_f = 25.4\%$

(all stresses in kg/cm^2)

ϵ %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_c}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1 / \bar{\sigma}_3$	Δu	$\frac{\Delta u}{\bar{\sigma}_c}$	A	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$
0	0	0	4.00	4.00	1.00	0	0	—	0	4.00
.02	.29	.072	3.87	4.16	1.07	.13	.032	.45	.145	4.015
.086	.843	.211	3.50	4.343	1.24	.50	.125	.59	.421	3.921
.216	1.706	.426	3.02	4.726	1.56	.98	.245	.57	.853	3.873
.28	1.863	.466	2.78	4.643	1.67	1.22	.305	.65	.931	3.711
.43	2.131	.533	2.53	4.661	1.84	1.47	.367	.69	1.065	3.595
.59	2.248	.562	2.34	4.588	1.96	1.66	.415	.74	1.129	3.464
.80	2.347	.587	2.13	4.477	2.10	1.87	.467	.80	1.173	3.303
1.06	2.404	.601	1.92	4.324	2.23	2.08	.52	.87	1.202	3.122
1.39	2.430	.607	1.72	4.15	2.41	2.28	.57	.94	1.215	2.935
2.12	2.440	.610	1.48	3.92	2.65	2.52	.63	1.03	1.220	2.70
2.34	2.460	.615	1.41	3.87	2.74	2.59	.647	1.05	1.230	2.64
2.59	2.460	.615	1.39	3.85	2.76	2.61	.652	1.06	1.230	2.62
2.77	2.460	.615	1.35	3.81	2.82	2.65	.662	1.08	1.230	2.58
4.04	2.325	.581	1.17	3.495	2.98	2.83	.707	1.21	1.162	2.332
5.12	2.385	.596	1.09	3.475	3.19	2.91	.727	1.22	1.192	2.282
6.12	2.344	.586	1.05	3.394	3.23	2.95	.737	1.26	1.172	2.222
6.44	2.374	.594	1.03	3.404	3.31	2.97	.742	1.25	1.187	2.217
8.84	2.310	.577	.98	3.29	3.36	3.02	.755	1.31	1.155	2.135
10.70	2.308	.577	.93	3.238	3.48	3.07	.767	1.33	1.154	2.084
11.50	2.288	.572	.92	3.208	3.48	3.08	.77	1.35	1.144	2.064
13.34	2.211	.553	.91	3.121	3.43	3.09	.772	1.40	1.105	2.015

Soil Properties

Test No. $\overline{CIUC}$-1 Tested by J.V. $\bar{\sigma}_c = 6.00$ kg/cm²

Batch No. S-5 $d\varepsilon/dt$ = 1cm/600min P.P.R. not measured

8 vertical paper drains back pressure 2.00 kg/cm²

σ_1 and σ_3 varied t_c = 3 days

$\bar{\sigma}_c$ Kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	81.20	8.00	10.15	30.4	94.2	.893	—	1.915
2.00	76.99	—	—	26.9	94.6	.794	4.7	1.97
4.00	73.62	—	—	24.6	95.3	.738	8.2	2.02
6.00	70.55	7.59	9.42	23.45	100	.655	12.6	2.08

circ. at end of test : 4.550 in.

Stress – Strain Data

Test No. $\overline{CIUC}$-1   Tested by J.V.   7/1963 $\bar{\sigma}_c = 6.00$ kg/cm² $w_i = 30.4\%$
$w_f = 23.45\%$

(all stresses in kg/cm²)

ϵ * %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_c}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1 / \bar{\sigma}_3$	Δu	$\frac{\Delta u}{\bar{\sigma}_c}$	A	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$
0	0	0	6.00	6.00	1.00	0	0	—	0	6.00
.16	1.02	.17	5.41	6.43	1.19	.59	.098	.57	.501	5.911
.195	1.365	.228	5.14	6.505	1.25	.86	.143	.63	.682	5.822
.25	1.598	.266	4.92	6.518	1.32	1.08	.18	.67	.805	5.725
.29	1.971	.329	4.64	6.111	1.425	1.36	.226	.69	.992	5.632
.35	2.253	.376	4.35	6.603	1.51	1.65	.275	.728	1.135	5.485
.52	2.665	.444	3.67	6.335	1.72	2.33	.388	.86	1.34	5.01
.726	2.965	.494	3.05	6.015	1.97	2.95	.492	.985	1.482	4.532
1.005	3.03	.505	2.80	5.83	2.08	3.20	.533	1.05	1.515	4.315
1.14	3.085	.514	2.64	5.725	2.16	3.36	.56	1.09	1.542	4.182
1.33	3.13	.522	2.50	5.63	2.25	3.50	.584	1.12	1.565	4.065
1.49	3.135	.523	2.33	5.465	2.34	3.67	.612	1.17	1.567	3.897
3.50	3.149	.525	1.67	4.819	2.88	4.33	.722	1.37	1.574	3.244
3.93	3.143	.524	1.59	4.733	2.98	4.41	.735	1.40	1.571	3.161
4.42	3.118	.52	1.54	4.658	3.02	4.46	.744	1.43	1.554	3.094
5.06	3.116	.519	1.43	4.546	3.18	4.57	.761	1.47	1.558	2.988
5.56	3.103	.517	1.40	4.503	3.22	4.60	.767	1.48	1.501	2.901
6.04	3.072	.512	1.39	4.462	3.21	4.61	.768	1.50	1.536	2.926
6.53	3.072	.512	1.38	4.452	3.22	4.62	.771	1.50	1.536	2.916
7.99	3.07	.511	1.29	4.36	3.38	4.71	.785	1.53	1.535	2.825
8.61	3.028	.505	1.28	4.308	3.36	4.72	.787	1.56	1.514	2.794
9.10	3.036	.505	1.28	4.316	3.37	4.72	.787	1.56	1.518	2.798
10.10	2.998	.50	1.23	4.228	3.44	4.77	.796	1.59	1.499	2.729
10.85	2.981	.497	1.23	4.211	3.42	4.77	.796	1.60	1.490	2.72

* subtract 0.06% ϵ to correct for seating error

Soil Properties

Test No. $\overline{CIUC}$ - 3   Tested by H. A. B. R. 10/1963 $\bar{\sigma}_c$ = 8.00 kg/cm²

Batch No. S-5 $d\varepsilon/dt$ = 1cm/600 min   P. P. R. = 85 %

8 vertical paper drains   back pressure 3.00 kg/cm²

σ_1 and σ_3 varied t_c = 3 days

$\bar{\sigma}_c$ Kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	80.80	8.00	10.10	31.5	94.51	.925	—	1.885
2.00	76.22	—	—	27.9	94.7	.816	5.7	1.96
4.00	72.12	—	—	24.6	94.9	.723	10.5	2.02
8.00	68.45	7.38	9.31	23.0	100	.641	14.8	2.10

circ. at end of test : 4.467 in.

Stress – Strain Data

Test No. $\overline{\text{CIUC}}$ – 3 Tested by H. A. B. R. 10/1963 $\bar{\sigma}_c = 8.00\ kg/cm^2$ $w_i = 31.5\%$
$w_f = 23.0\%$

(all stresses in kg/cm^2)

ε * %	$\bar{\sigma}_1 - \bar{\sigma}_3$ corrected	$\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_c}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1 / \bar{\sigma}_3$	Δu	$\frac{\Delta u}{\bar{\sigma}_c}$	A	$\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$
0	0	0	8.00	8.00	1.00	0	0	—	0	8.00
.05	.094	.017								
.14	.775	.097	7.60	8.375	1.10	+.40	.05	.52	.387	7.987
.21	2.25	.271	6.15	8.40	1.36	1.85	.231	.82	1.125	7.275
.31	2.82	.353	5.85	8.67	1.48	2.15	.269	.76	1.41	7.26
.45	3.38	.423	5.10	8.48	1.65	2.90	.362	.86	1.69	6.79
.62	3.72	.465	4.65	8.37	1.80	3.35	.419	.90	1.86	6.51
.76	3.91	.489	4.46	8.38	1.88	3.54	.442	.90	1.955	6.415
.92	4.10	.513	3.93	7.93	2.06	4.17	.521	1.02	2.05	5.88
1.04	4.20	.525	3.51	7.71	2.20	4.49	.561	1.07	2.10	5.61
1.21	4.27	.534	3.37	7.64	2.26	4.63	.579	1.08	2.135	5.505
1.38	4.33	.542	3.08	7.41	2.41	4.92	.615	1.14	2.165	5.245
1.91	4.42	.553	2.98	7.40	2.48	5.02	.627	1.14	2.21	5.19
2.07	4.45	.556	2.80	7.25	2.58	5.20	.65	1.17	2.225	5.025
2.59	4.50	.562	2.52	7.02	2.78	5.48	.686	1.22	2.25	4.77
3.19	4.51	.563	2.47	7.08	2.86	5.53	.692	1.23	2.255	4.725
3.79	4.59	.574	2.16	6.75	3.12	5.94	.73	1.27	2.295	4.455
4.65	4.55	.569	2.02	6.57	3.25	5.99	.748	1.32	2.275	4.295
5.00	4.51	.563	2.02	6.53	3.23	5.98	.748	1.33	2.255	4.275
5.35	4.44	.556	1.91	6.35	3.33	6.09	.761	1.37	2.22	4.13
6.15	4.56	.570	1.88	6.38	3.40	6.12	.765	1.35	2.28	4.16
6.90	4.52	.565	1.88	6.40	3.40	6.13	.766	1.36	2.26	4.14
7.60	4.55	.569	1.81	6.36	3.52	6.19	.774	1.36	2.275	4.085
8.26	4.59	.574	1.81	6.40	3.54	6.19	.774	1.35	2.295	4.105

(continued)

* subtract 0.1 % ε to correct for seating error

Test No. $\overline{\text{CIUC}}$-3

Stress - Strain Data
(continued)

(all stresses in kg/cm²)

ϵ %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_c}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1 / \bar{\sigma}_3$	Δu	$\frac{\Delta u}{\bar{\sigma}_c}$	A	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$
8.62	4.55	.568	1.80	6.35	3.52	6.20	.775	1.36	2.275	4.075
9.05	4.47	.559	1.84	6.27	3.42	6.16	.77	1.38	2.235	4.075
9.65	4.50	.563	1.80	6.30	3.50	6.20	.775	1.38	2.25	4.05
10.00	4.49	.562	1.77	6.26	3.54	6.23	.779	1.39	2.245	4.015
10.35	4.50	.563	1.77	6.27	3.54	6.23	.78	1.38	2.25	4.02
10.85	4.52	.565	1.74	6.22	3.51	6.24	.78	1.38	2.26	4.00

Soil Properties

Test No. $\overline{\text{CIUE}}$-1 Tested by J.V. 7/1963 $\bar{\sigma}_c = 4.00$ Kg/cm²

Batch No. S-5 stress controlled, 9 hrs to failure; without paper drains

back pressure 3.00 Kg/cm² P.P.R. not measured

$\Delta\sigma_a = 0$ and σ_r increased $t_c = 3$ days

$\bar{\sigma}_c$ Kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_0}$ %	γ_t g/cc
Initial	88.00	8.63	10.2	30.1	94.7	.885	—	1.93
1.00	84.32	—	—	29.1	100	.808	4.1	1.98
2.00	81.95	—	—	27.3	100	.756	6.85	2.01
4.00	78.65	8.50	9.25	24.6	100	.683	10.7	2.06

circ. at end of test : not measured

Stress - Strain Data

Test No. $\overline{\text{CIUE}}$-1   Tested by J. V. 7/1963 $\bar{\sigma}_c = 4.00$ kg/cm² $\omega_i = 30.1\%$
$\omega_f = 24.6\%$

(all stresses in kg/cm²)

ϵ %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_c}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\Delta u^{(3)}$	$\frac{\Delta u}{\bar{\sigma}_c}$ (3)	A	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$
0	0	0	4.00	4.00	1.00	0	0	—	0	4.00
0	.294	.073	3.806	4.10	1.08	.194	.048	.66	.147	3.953
.03	.592	.148	3.668	4.26	1.16	.33	.083	.56	.296	3.964
.06	.886	.222	3.234	4.12	1.27	.765	.191	.86	.443	3.677
.09	1.085	.272	3.005	4.09	1.36	1.00	.250	.92	542	3.547
.12	1.18	.295	2.86	4.04	1.41	1.14	.285	.97	.59	3.45
.16	1.282	.321	2.718	4.00	1.47	1.28	.320	1.00	.641	3.359
.22	1.38	.346	2.48	3.86	1.56	1.52	.380	1.10	.69	3.17
.36	1.48	.370	2.23	3.71	1.66	1.77	.443	1.19	.74	2.97
.85	1.68	.42	1.62	3.30	2.03	2.38	.595	1.42	.84	2.46
(2)1.32	1.735	.434	1.095	2.83	2.60	2.905	.726	1.67	.867	1.962
(4)2.23	1.742	.435	1.06	2.81	2.65	2.94	.735	1.69	.871	1.931
2.98	1.752	.438	.89	2.65	2.97	3.11	.778	1.77	.876	1.766
4.18	1.763	.441	.75	2.53	3.37	3.25	.813	1.84	.881	1.631
5.70	1.792	.448	.72	2.53	3.50	3.28	.820	1.83	.896	1.616
(1)8.72	1.854	.453	.596	2.44	4.16	3.414	.853	1.84	.927	1.513
10.18	1.885	.459	.555	2.44	4.40	3.445	.862	1.83	.942	1.397

(1) parabolic area correction applied beyond $\epsilon = 8.72\%$
(2) last load increase applied
(3) $\Delta u = \bar{\sigma}_c - \bar{\sigma}_3$
(4) necking started

Soil Properties

Test No. $\overline{\text{CIUE}}$ -2 Tested by J.V. 11/1963 $\bar{\sigma}_c = 4.00$

Batch No. S-6 $d\varepsilon/dt$ = 0.000048 inch/min P.P.R. = 95%

2 spiral paper drains back pressure 3.00 kg/cm²

$\Delta\sigma_r = 0$ and σ_a decreased $t_c = 12$ days

σ_c kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc			
Initial	87.40	8.64	10.10	30.9	94.3	.908	—	1.91			
1.38	82.95	—	—	27.8	95.2	.811	5.1	1.96			
3.00	78.73	—	—	24.8	96.0	.719	9.9	2.03			
4.00	76.10	8.38	9.10	23.75	100	.659	13.1	2.08			

circ. at end of test : 3.715 in.

Stress - Strain Data

Test No $\overline{\text{CIUE}}$-2   Tested by J.V. 11/1963 $\bar{\sigma}_c = 4.00$ kg/cm² $w_i = 30.9\%$
$w_f = 23.75\%$

(all stresses in kg/cm²)

ε %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_c}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1 / \bar{\sigma}_3$	$\Delta u^{(2)}$	$\frac{\Delta u}{\bar{\sigma}_c}$ (2)	A	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$
0	0	0	4.00	4.00	1.00	0	0	—	0	4.00
0	.544	.136	3.426	3.97	1.16	.575	.144	.69	.272	3.698
.13	1.702	.426	2.578	4.28	1.66	1.42	.355	.84	.851	3.429
.26	1.86	.465	2.24	4.10	1.83	1.76	.440	.95	.93	3.17
.44	2.04	.511	1.79	3.83	2.14	2.21	.552	1.08	1.02	2.81
.53	2.085	.522	1.695	3.78	2.23	2.305	.576	1.105	1.042	2.737
.99	2.110	.529	1.26	3.37	2.68	2.74	.685	1.30	1.055	2.315
1.24	2.105	.528	1.165	3.27	2.80	2.835	.709	1.35	1.052	2.217
1.52	2.10	.526	1.05	3.15	3.00	2.95	.737	1.40	1.05	2.10
2.92	2.069	.518	.691	2.76	4.00	3.31	.828	1.60	1.034	1.725
3.43	2.039	.511	.721	2.76	3.83	3.28	.820	1.61	1.019	1.740
3.60	2.044	.512	.676	2.72	4.02	3.325	.831	1.62	1.022	1.698
4.10	2.03	.509	.67	2.70	4.03	3.33	.833	1.64	1.015	1.685
4.63	2.035	.510	.635	2.67	4.20	3.365	.841	1.65	1.017	1.652
4.95	2.03	.509	.60	2.63	4.37	3.40	.850	1.67	1.015	1.615
6.12	2.05	.512	.57	2.62	4.60	3.43	.857	1.67	1.025	1.595
(1)7.86	2.03	.507	.57	2.60	4.57	3.43	.857	1.69	1.015	1.585
8.40	2.00	.500	.58	2.58	4.45	3.42	.855	1.71	1.00	1.58
9.99	1.83	.458	.64	2.47	3.86	3.36	.865	1.83	.915	1.555
10.20	1.793	.449	.637	2.43	3.82	3.363	.866	1.87	.896	1.533
10.80	1.763	.441	.627	2.39	3.81	3.373	.868	1.91	.881	1.508
11.14	1.743	.436	.637	2.38	3.74	3.363	.866	1.93	.871	1.508

(1) parabolic area correction applied beyond $\varepsilon = 7.48\%$
(2) $\Delta u = \bar{\sigma}_c - \bar{\sigma}_3$

Soil Properties

Test No. $\overline{\text{CIUE}}$ - 3   Tested by J.V.   11/1963 $\bar{\sigma}_c = 6.00$ kg/cm^2

Batch No. S-6 $d\epsilon/dt = 0.000048$ inch/min.   P.P.R. = 98 %

2 spiral paper drains   back pressure 3.00 kg/cm^2

$\Delta\sigma_r = 0$ and σ_a decreased $t_c = 6$ days

$\bar{\sigma}_c$ kg/cm^2	Volume cc	Length cm	Area cm^2	w %	S %	e	$\frac{\Delta e}{1+e_0}$ %	γ_t g/cc
Initial	87.70	8.64	10.15	29.0	93.5	.870	—	1.93
2.00	83.01	—	—	26.0	94.4	.763	5.7	1.98
4.00	78.91	—	—	23.2	95.5	.676	10.4	2.09
6.00	75.66	8.31	9.10	21.6	100	.606	14.1	2.11

circ. at end of test : 3.323 in

Stress - Strain Data

Test No. $\overline{CIUE}$-3 Tested by J.V. 11/1963 $\bar{\sigma}_c = 6.00$ kg/cm² $w_i = 29.0$ %
$w_f = 21.6$ %

(all stresses in kg/cm²)

ϵ * %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_c}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu	$\frac{\Delta u}{\bar{\sigma}_c}$	A	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$
0	0	0	6.00	6.00	1.00	0	0	—	0	6.00
.28	2.185	.364	3.755	5.94	1.58	2.245	.374	1.03	1.098	4.848
.33	2.285	.381	3.515	5.80	1.65	2.485	.415	1.09	1.143	4.658
.336	2.295	.382	3.545	5.84	1.65	2.455	.409	1.07	1.148	4.693
.593	2.62	.436	3.06	5.68	1.85	2.94	.490	1.12	1.31	4.370
2.16	2.73	.455	1.38	4.11	2.98	4.62	.77	1.69	1.36	2.75
2.22	2.71	.451	1.38	4.09	2.97	4.62	.77	1.70	1.31	2.74
2.84	2.72	.453	1.18	3.90	3.30	4.82	.805	1.77	1.31	2.54
2.94	2.72	.453	1.17	3.89	3.33	4.83	.805	1.84	1.31	2.53
3.72	2.73	.455	1.11	3.84	3.45	4.89	.815	1.85	1.365	2.48
4.39	2.732	.456	.978	3.71	3.80	5.02	.836	1.86	1.366	2.344
4.82	2.742	.457	.928	3.67	3.95	5.07	.845	1.855	1.371	2.249
5.15	2.752	.458	.908	3.66	4.03	5.09	.847	1.87	1.376	2.284
5.50	2.757	.459	.913	3.67	4.02	5.087	.847	1.87	1.379	2.291
6.19	2.758	.460	.842	3.60	4.26	5.158	.86	1.86	1.379	2.221
6.41	2.748	.458	.852	3.60	4.13	5.148	.858	1.86	1.374	2.226
6.99	2.738	.456	.872	3.61	4.13	5.128	.856	1.875	1.369	2.241
(1) 7.30	2.808	.468	.802	3.61	4.49	5.198	.866	1.850	1.404	2.206
7.74	2.818	.469	.792	3.61	4.56	5.208	.868	1.85	1.409	2.201
9.40	2.734	.455	.836	3.57	4.27	5.164	.862	1.882	1.367	2.263
9.62	2.584	.431	.906	3.57	3.94	5.094	.85	1.972	1.292	2.238
10.36	2.575	.429	.925	3.50	3.89	5.075	.846	1.972	1.288	2.212
12.02	2.288	.382	1.022	3.31	3.24	4.978	.83	2.180	1.144	2.166
13.10	2.168	.362	1.032	3.20	3.10	4.968	.828	2.28	1.084	2.116
15.25	1.99	.331	1.07	3.06	2.86	[illegible]	.823	2.48	.995	2.065

(1) parabolic area correction applied beyond $\epsilon = 7.30$%
* probably some seating error

Soil Properties

Test No. $\overline{C(K_o)UC}$ - 1   Tested by J. V. $\bar{\sigma}_{1c}$ = 4.10 kg/cm² $\bar{\sigma}_{3c}$ = 2.16 kg/cm²

Batch No. S-5 $d\epsilon/dt$ = 600 min/1cm   8 vertical paper drains

back pressure 3.00 Kg/cm²   P.P.R. not measured

σ_1 and σ_3 varied t_c = 6 days

$\bar{\sigma}_{1c}$ kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	80.80	8.00	10.10	31.2	95.3	.91	—	1.915
2.17	75.37	—	—	28.2	100	.782	6.7	2.01
3.27	72.82	—	—	26.6	100	.735	9.2	2.02
4.10	72.42	7.42	9.76	25.9	100	.72	10.0	2.04

circ. at end of test: 4.788 in.

consolidated against back pressure

Stress – Strain Data

Test NO. $\overline{C(K_o)UC}$ - 1 Tested by J. V. 7/1963 $\bar{\sigma}_{1c} = 4.10\ kg/cm^2$ $\bar{\sigma}_{3c} = 2.16\ kg/cm^2$

$w_i = 31.2\ \%$ $w_f = 25.9\ \%$

(all stresses in kg/cm^2)

ε %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu	$\frac{\Delta u}{\bar{\sigma}_{1c}}$	$A^{(1)}$	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$	$\frac{\Delta\sigma_1 - \Delta\sigma_3}{\bar{\sigma}_{1c}}$
0	1.94	.473	2.16	4.10	1.90	0	0	—	.97	3.13	0
.034	2.33	.568	2.05	4.38	2.14	.11	.027	.28	1.165	3.215	.095
.068	2.52	.615	1.93	4.45	2.30	.23	.056	.40	1.26	3.19	.142
.105	2.60	.632	1.86	4.46	2.40	.30	.073	.45	1.30	3.16	.159
.205	2.69	.656	1.70	4.39	2.58	.46	.112	.61	1.345	3.045	.183
.274	2.695	.657	1.69	4.385	2.59	.47	.114	.62	1.347	3.037	.184
.445	2.665	.65	1.68	4.345	2.58	.48	.117	.66	1.332	3.012	.177
.514	2.66	.649	1.58	4.24	2.68	.58	.141	.81	1.33	2.91	.176
.583	2.64	.643	1.58	4.22	2.67	.58	.141	.83	1.32	2.90	.170
.68	2.61	.636	1.55	4.16	2.68	.61	.149	.87	1.305	2.855	.163
.86	2.56	.624	1.50	4.06	2.70	.66	.161	.99	1.28	2.78	.151
1.05	2.52	.614	1.44	4.00	2.78	.72	.175	1.16	1.26	2.72	.141
1.20	2.48	.605	1.41	3.89	2.76	.75	.183	1.21	1.24	2.65	.132
1.37	2.44	.595	1.41	3.85	2.73	.75	.183	1.39	1.22	2.63	.122
1.54	2.41	.588	1.40	3.81	2.72	.76	.185	1.52	1.205	2.60	.115
1.71	2.40	.585	1.24	3.64	2.94	.92	.224	1.96	1.20	2.44	.112
1.88	2.375	.579	1.23	3.605	2.93	.93	.227	2.02	1.187	2.417	.106
2.06	2.34	.571	1.21	3.55	2.94	.95	.232	2.18	1.17	2.38	.098
2.22	2.32	.566	1.18	3.50	2.97	.98	.239	2.44	1.16	2.34	.093
2.40	2.29	.558	1.15	3.44	2.99	1.01	.246	2.66	1.145	2.295	.085
2.57	2.275	.555	1.14	3.415	2.99	1.02	.249	2.91	1.137	2.277	.082
2.74	2.26	.551	1.11	3.37	3.04	1.05	.256	3.13	1.13	2.24	.078
2.91	2.23	.544	1.10	3.33	3.03	1.06	.258	3.31	1.11	2.21	.071
3.08	2.217	.54	1.08	3.297	3.04	1.08	.263	3.90	1.108	2.188	.068
3.42	2.19	.534	1.07	3.26	3.04	1.09	.266	4.36	1.095	2.165	.061

(1) $A = \dfrac{\Delta u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}$

(continued)

Test No. $\overline{C(K_o)UC}$-1

Stress – Strain Data
(continued)

ϵ %	$\bar\sigma_1 - \bar\sigma_3$ corrected	$\frac{\bar\sigma_1 - \bar\sigma_3}{\bar\sigma_{1c}}$	$\bar\sigma_3$	$\bar\sigma_1$	$\bar\sigma_1 / \bar\sigma_3$	Δu	$\frac{\Delta u}{\bar\sigma_{1c}}$	A	$\frac{\bar\sigma_1 - \bar\sigma_3}{2}$	$\frac{\bar\sigma_1 + \bar\sigma_3}{2}$	$\frac{\Delta\sigma_1 - \Delta\sigma_3}{\bar\sigma_{1c}}$
3.60	2.182	.531	1.04	3.222	3.10	1.12	.273	4.63	1.09	2.13	.059
5.85	2.00	.488	.90	2.90	3.22	1.26	.307	21.00	1.00	1.90	.015
6.17	1.96	.478	.90	2.86	3.18	1.26	.307	62.80	.98	1.88	.004
6.52	1.95	.475	.88	2.83	3.22	1.25	.312	128.0	.975	1.855	.00
6.65	1.94	.473	.85	2.79	3.28	1.31	.320	∞	.97	1.82	.000
7.20	1.92	.468	.86	2.78	3.24	1.30	.317	-65.0	.96	1.82	.004
7.54	1.90	.463	.84	2.74	3.26	1.32	.322	-33.0	.95	1.79	.010
7.89	1.88	.458	.84	2.72	3.24	1.32	.322	-22.0	.94	1.78	.014
8.22	1.87	.456	.83	2.70	3.25	1.33	.324	-19.0	.93	1.76	.017
8.73	1.85	.451	.82	2.67	3.2[illegible]	1.34	.327	-14.9	.92	1.74	.022
9.25	1.84	.449	.80	2.64	3.30	1.36	.332	-13.6	.92	1.72	.024
10.60	1.783	.435	.79	2.573	3.26	1.37	.334	-9.56	.89	1.68	.038
10.98	1.758	.429	.77	2.528	3.29	1.39	.339	-7.73	.879	1.649	.044
11.30	1.743	.425	.76	2.503	3.29	1.40	.342	-7.0	.871	1.631	.048
12.00	1.723	.420	.76	2.483	3.26	1.40	.342	-6.35	.861	1.621	.053

Soil Properties

Test No. $\overline{C(K_o)UC}$-2   Tested by N.B.   11/1963 $\bar{\sigma}_{1c} = 6.10$ kg/cm² $\bar{\sigma}_{3c} = 3.27$ kg/cm²

Batch No. S-6 $d\varepsilon/dt$ = 1cm/600 min   P.P.R. = 90%

8 vertical paper drains   back pressure 3.00 kg/cm²

σ_1 and σ_3 varied t_c = 8 days

$\bar{\sigma}_{1c}$ kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_0}$ %	γ_t g/cc
Initial	81.20	8.00	10.20	29.7	95.5	.860	—	1.93
2.03	78.05	—	—	27.4	96.3	.786	4.0	1.98
3.15	74.89	—	—	25.0	97.2	.715	7.8	2.02
6.10	71.97	7.40	9.72	23.3	100	.648	11.4	2.08

circ. at end of test: 4.787 in

Stress-Strain Data

Test No. $\overline{C(K_o)UC-2}$ Tested by N.B. 11/1963 $\bar{\sigma}_{1c} = 6.10\ kg/cm^2$ $\bar{\sigma}_{3c} = 3.27\ kg/cm^2$

$w_i = 30.3\ \%$ $w_f = 23.3\ \%$

(all stresses in kg/cm^2)

ϵ %	$\sigma_1 - \sigma_3$ Corrected	$\frac{\sigma_1-\sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu	$\frac{\Delta u}{\bar{\sigma}_{1c}}$	A	$\frac{\sigma_1-\sigma_3}{2}$	$\frac{\bar{\sigma}_1+\bar{\sigma}_3}{2}$
0	2.83	.463	3.27	6.10	1.865	0	0	—	1.47	4.74
.01	3.22	.528	3.30	6.52	1.98	-.03	-.005	.07	1.66	4.91
.03	3.49	.571	3.06	6.55	2.14	+.21	+.034	.32	1.75	4.81
.068	3.66	.600	3.07	6.73	2.19	.20	.033	.24	1.83	4.90
.103	3.81	.625	2.90	6.71	2.32	.37	.061	.278	1.91	4.81
.137	3.88	.636	2.87	6.75	2.35	.40	.065	.33	1.94	4.81
.22	3.97	.650	2.70	6.67	2.47	.57	.093	.50	1.99	4.67
.274	3.98	.652	2.68	6.66	2.48	.59	.097	.513	1.99	4.68
.309	4.00	.656	2.64	6.64	2.51	.63	.103	.538	2.00	4.64
.343	3.99	.654	2.61	6.60	2.53	.66	.108	.569	1.995	4.605
.43	4.00	.656	2.49	6.39	2.57	.78	.128	.657	2.00	4.49
.51	3.96	.649	2.44	6.30	2.58	.83	.136	.735	1.98	4.42
.58	3.84	.63	2.40	6.24	2.60	.87	.142	.861	1.92	4.32
.70	3.89	.638	2.27	6.16	2.71	1.00	.164	.943	1.945	4.215
.80	3.87	.634	2.19	6.06	2.76	1.08	.177	1.04	1.935	4.125
.87	3.84	.629	2.13	5.97	2.80	1.14	.187	1.13	1.92	4.05
.94	3.81	.625	2.12	5.93	2.80	1.15	.188	1.17	1.905	4.025
1.03	3.79	.621	2.05	5.84	2.85	1.22	.200	1.27	1.895	3.945
1.16	3.74	.613	1.99	5.73	2.88	1.28	.210	1.41	1.87	3.86
1.24	3.72	.61	1.96	5.68	2.90	1.31	.215	1.47	1.86	3.82
1.37	3.68	.603	1.92	5.60	2.92	1.35	.221	1.59	1.84	3.76
1.51	3.63	.595	1.87	5.50	2.94	1.40	.229	1.75	1.815	3.685
1.66	3.59	.588	1.82	5.41	2.97	1.45	.238	1.91	1.795	3.615
1.73	3.58	.587	1.78	5.36	3.01	1.49	.244	1.99	1.79	3.57
1.87	3.56	.583	1.74	5.30	3.05	1.53	.251	2.10	1.78	3.52

(continued)

Test No. $\overline{C(K_o)}UC-2$

Stress – Strain Data (continued)

(all stresses in kg/cm^2)

ϵ %	$\sigma_1-\sigma_3$ corrected	$\frac{\sigma_1-\sigma_3}{\bar\sigma_{1c}}$	$\bar\sigma_3$	$\bar\sigma_1$	$\bar\sigma_1/\bar\sigma_3$	Δu	$\frac{\Delta u}{\bar\sigma_{1c}}$	A	$\frac{\sigma_1-\sigma_3}{2}$	$\frac{\bar\sigma_1+\bar\sigma_3}{2}$
1.98	3.51	.575	1.69	5.20	3.07	1.58	.259	2.32	1.76	3.45
2.16	3.48	.570	1.63	5.11	3.14	1.64	.269	2.52	1.74	3.37
2.34	3.45	.566	1.57	5.02	3.20	1.70	.279	2.74	1.73	3.30
2.50	3.41	.559	1.55	4.96	3.20	1.72	.282	2.97	1.705	3.255
2.68	3.38	.554	1.51	4.89	3.24	1.76	.288	3.20	1.69	3.20
2.81	3.36	.550	1.50	4.86	3.24	1.77	.29	3.34	1.68	3.18
3.00	3.32	.544	1.46	4.78	3.28	1.81	.297	3.69	1.66	3.12
3.18	3.29	.539	1.45	4.74	3.26	1.82	.298	3.96	1.645	3.095
3.52	3.25	.533	1.38	4.63	3.35	1.89	.310	4.50	1.625	3.005
3.79	3.22	.527	1.32	4.54	3.43	1.95	.320	5.00	1.61	2.93
3.96	3.18	.521	1.30	4.48	3.44	1.97	.323	5.62	1.59	2.89
4.08	3.16	.518	1.27	4.43	3.48	2.00	.328	6.06	1.58	2.85
4.36	3.13	.513	1.23	4.36	3.54	2.04	.334	6.80	1.565	2.795
4.52	3.10	.508	1.22	4.32	3.54	2.05	.336	7.58	1.55	2.77
4.77	3.08	.505	1.20	4.28	3.57	2.07	.339	8.30	1.54	2.74
4.94	3.06	.501	1.16	4.22	3.64	2.11	.346	9.17	1.53	2.69
5.09	3.04	.498	1.16	4.20	3.63	2.11	.346	10.05	1.52	2.68
5.41	2.99	.49	1.16	4.15	3.57	2.11	.346	13.20	1.495	2.655
5.92	2.95	.483	1.13	4.08	3.61	2.19	.351	17.80	1.475	2.605
6.37	2.89	.474	1.07	3.96	3.70	2.20	.36	36.70	1.445	2.515
6.90	2.85	.467	1.06	3.91	3.69	2.21	.362	110.50	1.425	2.485
7.58	2.79	.457	1.05	3.84	3.65	2.22	.364	-55.5	1.395	2.445
8.35	2.73	.455	1.01	3.74	3.70	2.26	.370	-22.6	1.365	2.375
9.28	2.60	.426	1.01	3.61	3.58	2.26	.370	-9.83	1.300	2.31
9.78	2.57	.421	1.03	3.60	3.48	2.24	.367	-8.62	1.285	2.315
10.30	2.50	.410	.98	3.48	3.55	2.29	.376	-6.94	1.250	2.23
11.06	2.41	.395	.96	3.37	3.52	2.31	.379	-5.50	1.205	2.165
11.50	2.37	.389	.96	3.33	3.47	2.31	.379	-5.02	1.155	2.145
12.12	2.33	.382	.95	3.28	3.45	2.32	.36	-4.64	1.165	2.115

Soil Properties

Test No. $C(K_o)-\overline{UUC}-1$ T ~~ed by J.V. 7/1963 $\bar{\sigma}_{1c} = 4.08$ kg/cm² $\bar{\sigma}_{3c} = 2.16$ kg/cm²

Batch No. S-5 . $\Delta t = 600$ min/10 mm P.P.R. = 100 %

8 vertical paper drains back pressure 3.00 kg/cm²

σ_1 and σ_3 varied $t_c = 8$ days

$\bar{\sigma}_{1c}$ kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	86.60	8.63	10.05	31.3	96.6	.90	—	1.915
2.17	83.70	—	—	29.6	97.2	.85	2.6	1.945
3.28	81.69	—	—	28.2	97.7	.805	5.0	1.97
4.08	79.06	8.25	9.60	26.5	100	.74	8.4	2.02

circ. at end of test : not measured

Stress – Strain Data

Test No. $C(K_o)$-$\overline{UUC}$-1   Tested by J.V. 7/1963 $\bar{\sigma}_{1c} = 4.08$ kg/cm² $\bar{\sigma}_{3c} = 2.16$ kg/cm²

$w_i = 31.3\%$ $w_f = 26.5\%$ P.P.R. = 100%

(all stresses in kg/cm²)

ε %	$\sigma_1-\sigma_3$ corrected	$\frac{\sigma_1-\sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu	A	$\frac{\sigma_1-\sigma_3}{2}$	$\frac{\bar{\sigma}_1+\bar{\sigma}_3}{2}$	$\frac{\Delta u}{\bar{\sigma}_{1c}}$
−.31	1.92	.47	2.16	4.08	1.89	0	—	.96	3.12	0
−.26	.77	.189	2.52	3.29	1.31	.36	.313	.385	2.905	.088
−.15	.42	.103	2.55	2.97	1.16	.39	.26	.21	2.76	.095
−.09	.17	.04	2.52	2.69	1.07	.36 (1)	.20 (2)	.085	2.605	.088
0	0	0	2.52	2.52	1.00	.36/0.0	.187/–	0	2.52	.088
.05	.58	.142	2.20	2.78	1.26	.32	.551	.29	2.49	.079
.12	1.26	.309	1.90	3.16	1.66	.62	.488	.63	2.53	.152
.22	1.90	.465	1.63	3.53	2.16	.89	.468	.95	2.58	.218
.34	2.10	.514	1.60	3.70	2.31	.92	.434	1.05	2.65	.225
.65	2.24	.549	1.44	3.68	2.56	1.08	.482	1.12	2.56	.264
.89	2.23	.547	1.37	3.60	2.63	1.19	.533	1.115	2.49	.292
1.17	2.18	.534	1.25	3.43	2.74	1.27	.582	1.09	2.34	.311
1.48	2.11	.517	1.17	3.28	2.80	1.35	.639	1.055	2.23	.331
1.76	2.07	.507	1.13	3.20	2.83	1.39	.671	1.035	2.17	.341
2.78	1.94	.475	.97	2.91	2.99	1.55	.798	.97	1.94	.380
3.25	1.89	.463	.95	2.84	2.99	1.57	.879	.945	1.89	.384
3.84	1.85	.453	.85	2.70	3.18	1.67	.902	.925	1.77	.409
4.41	1.81	.443	.82	2.63	3.21	1.70	.938	.905	1.72	.416
4.81	1.79	.438	.79	2.58	3.27	1.73	.966	.895	1.69	.423
7.37	1.66	.406	.71	2.37	3.34	1.81	1.091	.83	1.54	.443
9.03	1.60	.392	.69	2.29	3.32	1.83	1.143	.80	1.49	.448
9.40	1.58	.387	.67	2.25	3.36	1.88	1.190	.79	1.46	.461
9.72	1.57	.384	.65	2.22	3.42	1.87	1.192	.785	1.435	.458
12.18	1.49	.365	.63	2.12	3.36	1.89	1.269	.745	1.375	.463
12.67	1.49	.365	.62	2.11	3.40	1.90	1.276	.745	1.365	.465

(1) $\Delta u = 0$ at $(\sigma_1 - \sigma_3) = 0$ (2) $A = \dfrac{\Delta u - \Delta\sigma_3}{(\sigma_1 - \sigma_3)}$

Soil Properties

Test No. $C(K_o)-\overline{UUC}-2$ Tested by J.V. 7/1963 $\bar{\sigma}_{1c} = 6.15\ kg/cm^2$ $\bar{\sigma}_{3c} = 3.24\ kg/cm^2$

Batch No. S-5 $d\varepsilon/dt$ = 600 min / 10 mm P.P.R. = 100 %

8 vertical paper drains back pressure 3.00 kg/cm^2

σ_1 and σ_3 varied t_c = 7 days

$\bar{\sigma}_{1c}$ kg/cm^2	Volume cc	Length cm	Area cm^2	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	88.20	8.64	10.20	31.0	92.0	.935	—	1.80
2.72	84.42	—	—	28.7	92.8	.855	4.1	1.93
4.66	80.85	—	—	26.1	93.5	.777	8.2	1.97
6.15	77.05	8.13	9.50	24.9	100	.694	12.5	2.05

circ. at end of test : not measured

Stress – Strain Data

Test No. $C(K_o)-\overline{UUC}-2$ Tested by J.V. 7/1963 $\bar{\bar{\sigma}}_{1c} = 6.15\ kg/cm^2$ $\bar{\bar{\sigma}}_{3c} = 3.24\ kg/cm^2$

$w_i = 31.0\%$ $w_f = 24.9\%$ P.P.R. = 100%

(all stresses in kg/cm^2)

ε %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1-\sigma_3}{\bar{\bar{\sigma}}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu	$\frac{\Delta u}{\bar{\bar{\sigma}}_{1c}}$	A	$\frac{\sigma_1-\sigma_3}{2}$	$\frac{\bar{\sigma}_1+\bar{\sigma}_3}{2}$
-.24	2.91	.473	3.24	6.15	1.90	0	0	—	1.455	4.695
-.21	1.64	.266	3.53	5.17	1.46	.29	.047	.228	.82	4.35
-.15	.95	.154	3.73	4.68	1.26	.49	.08	.25	.475	4.205
-.09	.44	.072	3.78	4.22	1.11	.54	.088	.218	.22	4.00
-.06	.26	.042	3.93	4.19	1.07	.69	.112	.26	.13	4.06
-.05	.18	.029	4.08	4.26	1.04	.84	.136	.308	.09	4.17
-.04	.13	.021	4.03	4.16	1.03	.79	.128	.284	.065	4.095
-.01	.03	.004	3.96	3.99	1.01	.72	.117	.25	.015	3.975
0	0	0	3.96	3.96	1.00	.72/0	.117/0	.248/—	0	3.96
.05	.91	.148	3.45	4.36	1.26	.51	.083	.561	.46	3.91
.08	1.50	.244	3.16	4.66	1.47	.80	.13	.533	.75	3.91
.14	2.18	.354	2.88	5.06	1.76	1.08	.175	.495	1.09	3.97
.21	2.82	.458	2.66	5.48	2.06	1.30	.211	.461	1.41	4.07
.35	3.30	.537	2.50	5.80	2.32	1.46	.238	.442	1.65	4.15
.55	3.47	.563	2.34	5.81	2.48	1.62	.264	.467	1.74	4.08
.73	3.48	.566	2.20	5.68	2.58	1.76	.286	.506	1.74	3.94
1.83	3.26	.530	1.76	5.02	2.85	2.20	.358	.674	1.63	3.39
2.32	3.14	.510	1.61	4.75	2.95	2.35	.382	.748	1.57	3.18
2.97	3.04	.494	1.52	4.56	3.00	2.44	.397	.802	1.52	3.04
3.48	3.00	.486	1.40	4.40	3.14	2.56	.416	.852	1.50	2.90
3.94	2.95	.480	1.35	4.30	3.18	2.61	.424	.884	1.48	2.83
4.28	2.92	.475	1.31	4.23	3.23	2.35	.382	.769	1.46	2.77
6.42	2.74	.445	1.21	3.95	3.26	2.75	.447	1.003	1.37	2.58
8.16	2.64	.429	1.07	3.71	3.46	2.89	.470	1.096	1.32	2.39
8.93	2.54	.413	1.07	3.66	3.42	2.89	.470	1.117	1.30	2.37
11.50	2.45	.398	1.02	3.47	3.40	2.94	.478	1.200	1.22	2.24
11.93	2.44	.396	1.02	3.46	3.39	2.94	.478	1.204	1.22	2.24
12.24	2.42	.394	1.01	3.43	3.39	2.95	.48	1.219	1.21	2.22
12.70	2.41	.392	1.01	3.42	3.39	2.95	.48	1.223	1.20	2.21

Soil Properties

Test No. $\overline{C(K_o)}URE-1$

Batch No. S-4

Tested by J.V. 6/1963 $\bar{\sigma}_{1c} = 4.04$ kg/cm² $\bar{\sigma}_{3c} = 2.16$ kg/cm²

stress controlled, 10 hrs to failure; P.P.R. = 90 %

3 spiral paper drains back pressure 3.00 kg/cm²

$\Delta\sigma_a = 0$ and σ_r increased $t_c = 3$ days

$\bar{\sigma}_{1c}$ kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	86.50	8.65	10.00	30.1	97.6	.854	—	1.95
1.50	84.80	—	—	29.1	97.8	.820	1.8	1.96
2.72	83.73	—	—	28.2	98.3	.797	3.1	1.97
4.04	80.78	8.21	9.82	26.1	100	.732	6.6	2.02

circ. at end of test : 3.362 in.

Stress – Strain Data

Test No. $\overline{C(K_o)}$URE-1 Tested by J.V. 6/1963 $\bar{\sigma}_{1c} = 4.04$ kg/cm² $\bar{\sigma}_{3c} = 2.16$ kg/cm²

$w_i = 30.1\%$ $w_f = 26.1\%$

(all stresses in kg/cm²)

ϵ %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\bar{\sigma}_r/\bar{\sigma}_a$	Δu (3)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (3)	A (4)	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$	$\Delta\sigma_r - \Delta\sigma_a$
0	1.875	.464	2.16	4.035	.535	0	0	—	.937	3.097	0
.001	1.565	.388	2.19	3.755	.583	.285	.071	.92	.782	2.972	.31
.001	.958	.237	2.33	3.288	.710	.752	.186	.82	.479	2.809	.917
.05	.650	.161	2.36	3.01	.785	1.03	.254	.84	.325	2.685	1.225
.08	.326	.081	2.23	2.556	.873	1.484	.368	.96	.162	2.392	1.549
.17	0	0	2.12	2.12	1.00	1.92	.475	1.024	0	2.12	1.875
.49	.307	.076	1.88	1.573	1.195	2.467	.610	1.13	.153	1.726	2.182
.94	.617	.153	1.63	1.013	1.61	3.027	.749	1.212	.308	1.321	2.492
(2) 2.67	.940	.233	1.41	.474	2.98	3.566	.883	1.268	.470	.944	2.815
4.70	.958	.238	1.25	.305	3.78	3.735	.925	1.32	.479	.784	2.833
6.60	.980	.243	1.16	.194	5.98	3.846	.953	1.348	.490	.684	2.855
(1) 7.53	.990	.245	1.15	.160	7.19	3.88	.961	1.352	.495	.655	2.865
7.95	.996	.247	1.14	.144	7.91	3.896	.965	1.357	.498	.642	2.871
8.38	.995	.246	1.12	.115	9.74	3.925	.972	1.368	.497	.612	2.870
9.00	1.002	.248	1.10	.098	11.20	3.942	.976	1.371	.501	.599	2.877
10.60	1.013	.251	1.10	.087	12.63	3.953	.978	1.368	.506	.593	2.888
12.10	1.027	.254	1.12	.093	12.05	3.947	.977	1.36	.513	.606	2.902
14.20	1.047	.259	1.12	.073	15.30	3.967	.983	1.358	.523	.596	2.922
16.10	1.07	.265	1.12	.050	22.40	3.99	.988	1.36	.535	.585	2.942

(1) parabolic area correction applied beyond $\epsilon = 7.53\%$. Necking occured at $\epsilon = 7.5\%$

(2) last load increase applied

(3) $\Delta u = \bar{\sigma}_{ac} - \bar{\sigma}_a$

(4) $A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$

Soil Properties

Test No. $\overline{C(K_o)U}RE$-2 Tested by J.V. 7/1963 $\bar{\sigma}_{1c} = 4.03$ Kg/cm² $\bar{\sigma}_{3c} = 2.14$ kg/cm²

Batch No. S-5 stress controlled, 13 hrs to failure; no filter paper drain

back pressure 3.00 kg/cm² P.P.R. not measured

$\Delta\sigma_a = 0$ and σ_r increased $t_c = 3$ days

$\bar{\sigma}_{1c}$ Kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	86.90	8.60	10.10	30.35	96.5	.88	—	1.94
2.17	84.67	—	—	28.9	97.1	.83	2.7	1.96
3.27	82.04	—	—	27.0	97.4	.773	5.7	2.00
4.03	80.40	8.15	9.87	26.5	100	.74	7.5	2.02

circ. at end of test: 3.975 in.

Stress - Strain Data

Test No. $\overline{C(K_o)}$URE -2 Tested by J.V. 7/1963 $\bar{\sigma}_{1c} = 4.03$ kg/cm² $\bar{\sigma}_{3c} = 2.16$ kg/cm²

$w_i = 30.35\%$ $w_f = 26.5\%$

(all stresses in kg/cm²)

ϵ %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\bar{\sigma}_r / \bar{\sigma}_a$	$\Delta u^{(4)}$	$\frac{\Delta u}{\bar{\sigma}_{1c}}$	A	$\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$	$\Delta\sigma_r - \Delta\sigma_a$
0	1.87	.464	2.16	4.03	.535	0	0	—	.935	3.095	0
.001	1.57	.39	2.27	3.84	.59	.19	.047	.633	.785	3.055	.30
.005	1.31	.325	2.35	3.66	.64	.37	.092	.661	.655	3.005	.56
.01	.99	.246	2.37	3.36	.71	.67	.166	.762	.495	2.865	.88
.05	.67	.166	2.42	3.09	.784	.94	.233	.783	.335	2.755	1.20
.13	.34	.084	2.35	2.69	.87	1.34	.332	.870	.170	2.52	1.53
.25	0.00	0	2.29	2.29	1.00	1.74	.431	.931	0	2.29	1.87
.50	.30	.074	2.11	1.810	1.16	2.22	.55	1.042	.15	1.96	2.17
1.07	.596	.148	1.73	1.134	1.52	2.895	.718	1.172	.298	1.432	2.466
1.24	.689	.171	1.69	1.001	1.69	3.03	.751	1.184	.345	1.346	2.559
1.61	.778	.193	1.63	.852	1.92	3.18	.790	1.20	.389	1.241	2.648
2.45	.858	.213	1.52	.662	2.30	3.37	.835	1.235	.429	1.091	2.728
(2) 4.30	.868	.216	1.47	.602	2.45	3.43	.850	1.252	.434	1.036	2.738
(3) 5.60	.876	.217	1.40	.524	2.66	3.505	.870	1.27	.438	.962	2.756
6.58	.883	.219	1.35	.467	2.88	3.56	.885	1.293	.441	.908	2.753
(1) 7.80	.910	.226	1.33	.420	3.16	3.61	.895	1.294	.455	.875	2.787
8.78	.919	.228	1.32	.401	3.30	3.63	.900	1.300	.459	.860	2.789
10.00	.923	.229	1.30	.377	3.45	3.65	.906	1.307	.466	.843	2.793
10.90	.933	.232	1.29	.357	3.61	3.66	.910	1.305	.466	.823	2.803
11.70	.942	.234	1.29	.348	3.70	3.68	.915	1.310	.471	.819	2.812
12.62	.946	.235	1.28	.334	3.83	3.695	.917	1.312	.473	.807	2.816
14.20	.961	.238	1.25	.289	4.32	3.74	.928	1.32	.480	.769	2.831
14.60	.963	.239	1.26	.297	4.25	3.74	.926	1.32	.481	.778	2.833
15.40	.975	.242	1.32	.345	3.82	3.65	.913	1.292	.487	.832	2.845
16.20	.979	.243	1.27	.311	4.14	3.72	.923	1.305	.489	.800	2.849

(1) parabolic area correction applied beyond $\epsilon = 7.80\%$ (4) $\Delta u = \bar{\sigma}_{ac} - \bar{\sigma}_a$

(2) last load increase applied

(3) necking started

(continued)

Test No. $\overline{C(K_o)}URE-2$

Stress - Strain Data
(continued)

ϵ %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\bar{\sigma}_r / \bar{\sigma}_a$	Δu	$\frac{\Delta u}{\bar{\sigma}_{1c}}$	A	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$	$\Delta(\sigma_1 - \sigma_3)$
16.70	.987	.245	1.31	.323	4.07	3.71	.92	1.30	.493	.816	2.857
17.30	.990	.246	1.32	.330	4.00	3.70	.918	1.28	.495	.825	2.88
18.42	1.000	.248	1.33	.330	4.02	3.70	.918	1.28	.500	.830	2.87
19.40	1.010	.252	1.33	.320	4.15	3.71	.92	1.29	.505	.825	2.88

Soil Properties

Test No. $\overline{C(K_o)URE}$ -4   Tested by J.V.   10/1963 $\bar{\sigma}_{1c} = 4.00\ kg/cm^2$ $\bar{\sigma}_{3c} = 2.16\ kg/cm^2$

Batch No. S-6 $d\epsilon/dt = 0.000048$ inch/min.   P.P.R. = 90%

back pressure $3.00\ kg/cm^2$ 2 spiral filter paper

$\Delta\sigma_r = 0$ and σ_a decreased $t_c = 6$ days

$\bar{\sigma}_{1c}$ Kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	87.00	8.64	10.15	30.5	97.0	.875	—	1.93
1.5	84.50	—	—	29.3	97.6	.821	2.9	1.98
2.72	82.35	—	—	27.4	98.4	.774	5.4	2.00
4.00	80.08	8.10	9.97	26.0	100	.726	8.0	2.03

circ. at end of test : not measured

Stress - Strain Data

Test No. $\overline{C(K_o)URE-4}$ Tested by J.V. 10/1963 $\bar{\sigma}_{1c} = 4.00$ kg/cm² $\bar{\bar{\sigma}}_{3c} = 2.16$ kg/cm²

$w_i = 30.5\%$ $w_f = 26.0\%$

(all stresses in kg/cm²)

ϵ %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\bar{\sigma}}_r$	$\bar{\sigma}_a$	$\bar{\bar{\sigma}}_r/\bar{\sigma}_a$	Δu (2)	$\frac{\Delta u}{\bar{\bar{\sigma}}_{1c}}$ (2)	A	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$	$\frac{\Delta\sigma_r - \Delta\sigma_a}{\bar{\bar{\sigma}}_{1c}}$
0	1.845	.46	2.16	4.00	.54	0	0	—	.922	3.082	0
0	1.29	.322	2.25	3.54	.64	.46	.115	.80	.645	2.895	.138
.45	.27	.067	2.53	2.06	1.13	1.94	.485	.92	.135	2.195	.527
.54	.39	.097	2.28	1.89	1.21	2.11	.527	.945	.195	2.085	.557
.70	.527	.132	2.17	1.643	1.32	2.36	.590	.99	.263	1.906	.592
.78	.612	.153	2.15	1.538	1.40	2.46	.615	.99	.306	1.844	.613
1.08	.752	.188	2.13	1.378	1.55	2.62	.655	1.01	.376	1.754	.648
1.34	.819	.204	1.95	1.131	1.72	2.87	.716	1.08	.409	1.540	.664
2.18	.895	.224	1.76	.865	2.02	3.135	.784	1.14	.447	1.312	.684
2.66	.964	.241	1.70	.736	2.30	3.265	.815	1.16	.482	1.218	.701
3.10	1.037	.259	1.65	.613	2.70	3.385	.846	1.21	.518	1.131	.719
3.50	1.055	.264	1.61	.555	2.87	3.445	.861	1.18	.527	1.082	.724
4.36	1.088	.272	1.57	.482	3.27	3.52	.880	1.20	.544	1.026	.732
4.95	1.10	.275	1.56	.460	3.39	3.54	.885	1.20	.55	1.01	.735
5.70	1.118	.290	1.55	.432	3.60	3.57	.893	1.20	.559	.991	.740
(1) 6.84	1.147	.286	1.59	.443	3.62	3.555	.890	1.19	.573	1.016	.746
7.52	1.177	.294	1.55	.373	4.19	3.625	.906	1.20	.588	.961	.754
8.10	1.188	.297	1.56	.382	4.11	3.62	.905	1.19	.594	.976	.757
9.68	1.312	.326	1.67	.368	4.52	3.63	.907	1.155	.651	1.019	.786
10.00	1.320	.330	1.70	.380	4.48	3.62	.905	1.145	.66	1.04	.790
10.46	1.290	.322	1.68	.390	4.32	3.61	.903	1.15	.645	1.035	.782
11.00	1.280	.320	1.68	.400	4.20	3.60	.900	1.15	.64	1.04	.780
12.10	1.254	.314	1.68	.426	3.91	3.574	.893	1.15	.627	1.053	.774
13.10	1.213	.304	1.68	.467	3.57	3.533	.885	1.15	.606	1.073	.765
13.45	1.211	.303	1.66	.449	3.69	3.55	.888	1.16	.605	1.054	.763

(1) necking started

(2) $\Delta u = \bar{\bar{\sigma}}_{ac} - \bar{\sigma}_a$

Soil Properties

Test No. $\overline{C(K_o)U}RE$-5 Tested by J.V. 11/1963 $\bar{\sigma}_{1c} = 6.00\ kg/cm^2$ $\bar{\sigma}_{3c} = 3.24\ kg/cm^2$

Batch No. 5-6 $d\epsilon/dt = 0.000098$ inch/min. P.P.R. = 100%

back pressure 3.00 kg/cm^2 2 spiral paper drains

$\Delta\sigma_r = 0$ and σ_a decreased $t_c = 4$ days

$\bar{\sigma}_{1c}$ kg/cm^2	Volume cc	Length cm	Area cm^2	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc			
Initial	87.00	8.63	10.10	30.8	96.5	.883	—	1.91			
2.17	82.11	—	—	27.2	97.2	.776	5.7	1.99			
4.16	78.70	—	—	24.7	97.8	.703	9.6	2.04			
6.00	75.94	7.61	9.98	23.1	100	.644	12.7	2.09			

circ. at end of test : 3.820 in.

Stress-Strain Data

Test No. $\overline{C(K_o)}$URE-5 Tested by J.V. 11/1963 $\bar{\sigma}_{1c} = 6.00$ kg/cm² $\bar{\sigma}_{3c} = 3.24$ kg/cm²

$w_i = 30.8\%$ $w_f = 23.1\%$

(all stresses in kg/cm²)

ϵ %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1-\sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\bar{\sigma}_r/\bar{\sigma}_a$	Δu (1)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$	A	$\frac{\sigma_1-\sigma_3}{2}$	$\frac{\bar{\sigma}_1+\bar{\sigma}_3}{2}$	$\frac{\Delta\sigma_r-\Delta\sigma_a}{\bar{\sigma}_{1c}}$
0	2.76	.460	3.24	6.00	.54	0	0	—	1.38	4.62	0
.03	2.09	.348	3.36	5.45	.62	.55	.092	.81	1.045	4.405	.112
.04	1.70	.274	3.41	5.11	.67	.89	.148	.84	.85	4.26	.186
.08	1.19	.198	3.56	4.746	.75	1.254	.209	.80	.595	4.155	.262
.27	.06	.01	3.62	3.56	1.02	2.44	.407	.86	.03	3.59	.47
.49	.61	.101	3.48	2.87	1.21	3.13	.521	.93	.305	3.175	.561
.92	.80	.133	3.41	2.61	1.31	3.39	.565	.95	.40	3.01	.594
.94	1.10	.183	3.08	1.98	1.55	4.02	.670	1.04	.55	2.53	.644
1.93	1.31	.218	2.62	1.31	2.00	4.69	.783	1.15	.655	1.965	.679
2.62	1.358	.226	2.41	1.052	2.29	4.948	.825	1.20	.679	1.731	.687
3.08	1.39	.232	2.33	.94	2.48	5.06	.843	1.22	.695	1.635	.692
3.32	1.42	.237	2.29	.875	2.60	5.125	.855	1.23	.71	1.585	.697
4.70	1.473	.246	2.22	.747	2.93	5.253	.877	1.23	.736	1.483	.705
5.08	1.484	.248	2.21	.726	3.03	5.274	.880	1.24	.742	1.468	.707
5.67	1.504	.251	2.22	.716	3.09	5.284	.881	1.24	.752	1.468	.711
5.86	1.524	.254	2.22	.686	3.20	5.314	.885	1.24	.762	1.448	.715
7.28	1.605	.267	2.21	.605	3.62	5.395	.900	1.23	.802	1.407	.728
7.64	1.615	.269	2.23	.605	3.66	5.395	.900	1.23	.807	1.412	.729
8.13	1.628	.271	2.25	.621	3.64	5.379	.897	1.23	.814	1.435	.731
8.53	1.646	.274	2.26	.614	3.71	5.386	.898	1.22	.823	1.437	.734
10.02	1.708	.285	2.31	.602	3.86	5.398	.900	1.21	.854	1.456	.745
10.28	1.658	.277	2.31	.652	3.58	5.348	.892	1.21	.829	1.481	.736
10.44	1.638	.273	2.31	.672	3.45	5.328	.889	1.21	.819	1.491	.733
11.21	1.628	.271	2.31	.682	3.40	5.318	.887	1.21	.814	1.496	.731
12.58	1.572	.262	2.26	.688	3.28	5.312	.885	1.23	.786	1.474	.722
13.18	1.532	.256	2.24	.708	3.15	5.292	.883	1.23	.766	1.474	.715
13.70	1.492	.249	2.19	.698	3.13	5.302	.884	1.24	.746	1.444	.710

(1) $\Delta u = \bar{\sigma}_{ac} - \bar{\sigma}_a$

Soil Properties

Test No. $\overline{C(\frac{1}{K_o})UE}$ - 1 — Tested by J. V. 1/1964 — $\bar{\sigma}_{1c}$ = 4.00 kg/cm² — $\bar{\sigma}_{3c}$ = 2.010 kg/cm²

Batch No. S-6 — $d\epsilon/dt$ = 0.000048 inch/min — P. P. R. = 100 %

2 spiral paper drains — back pressure = 3.00 kg/cm²

$\Delta\sigma_r = 0$ and σ_a decreased — t_c = 6 days

$\bar{\sigma}_{1c}$ Kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	86.90	8.64	10.05	29.65	96.1	.86	—	1.94
1.50	82.27	—	—	26.2	96.4	.759	5.4	2.00
3.00	79.12	—	—	24.0	96.8	.692	9.0	2.04
4.00	76.97	8.98	8.67	23.0	100	.641	11.8	2.09

circ. at end of test : not measured

Stress – Strain Data

Test No. $\overline{C(\frac{1}{K_o})}UE-1$ Tested by J.V. 1/1964

$\bar{\sigma}_{1c} = 4.00$ kg/cm² $\bar{\sigma}_{3c} = 2.018$ kg/cm²
$w_i = 29.65\%$ $w_f = 23.0\%$

(all stresses in kg/cm²)

ϵ %	$\bar{\sigma}_1 - \bar{\sigma}_3$ corrected	$\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\Delta u^{(2)}$	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	$A^{(3)}$	$\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$	$\frac{\Delta\bar{\sigma}_1 - \Delta\bar{\sigma}_3}{\bar{\sigma}_{1c}}$
0	1.982	.496	4.00	2.018	1.98	0	0	—	.991	3.009	0
.002	2.345	.586	4.18	1.835	2.28	.18	.045	.504	1.172	3.007	.090
.008	2.48	.62	4.14	1.66	2.49	.36	.09	.718	1.24	2.90	.124
.05	2.608	.652	4.13	1.522	2.71	.50	.125	.792	1.304	2.826	.156
.23	2.63	.658	3.96	1.33	2.98	.69	.172	1.06	1.315	2.645	.162
.45	2.546	.637	3.79	1.244	3.05	.77	.192	1.375	1.273	2.517	.141
.54	2.53	.632	3.72	1.19	3.13	.83	.208	1.548	1.265	2.455	.136
.68	2.52	.630	3.66	1.14	3.21	.86	.22	1.631	1.26	2.40	.134
.89	2.48	.620	3.54	1.06	3.34	.96	.24	1.925	1.24	2.30	.124
1.93	2.365	.592	3.24	.875	3.70	1.14	.285	2.99	1.182	2.057	.096
2.16	2.334	.583	3.18	.846	3.76	1.17	.292	3.34	1.167	2.013	.087
2.38	2.339	.585	3.14	.801	3.92	1.22	.305	3.406	1.169	1.970	.089
2.64	2.301	.576	3.08	.779	3.96	1.24	.310	4.01	1.15	1.929	.080
2.98	2.299	.574	3.06	.781	3.95	1.24	.310	3.90	1.149	1.930	.078
3.25	2.304	.577	3.08	.716	3.97	1.24	.310	3.86	1.152	1.928	.079
4.23	2.243	.561	3.08	.837	3.68	1.18	.295	4.53	1.121	1.958	.065
4.54	2.218	.555	3.04	.822	3.69	1.20	.300	5.06	1.109	1.931	.059
4.90	2.203	.551	2.98	.777	3.83	1.24	.310	5.61	1.101	1.878	.055
5.29	2.198	.549	2.97	.772	3.84	1.24	.310	5.77	1.099	1.871	.053
5.70	2.163	.541	2.97	.807	3.68	1.21	.302	6.69	1.081	1.888	.045
(1) 6.86	2.015	.504	2.82	.805	3.50	1.21	.302	36.80	1.007	1.812	.008
7.14	2.00	.500	2.77	.77	3.59	1.25	.312	69.20	1.00	1.77	.004
7.44	1.975	.494	2.75	.775	3.55	1.24	.310	−177.8	.987	1.762	−.002
7.56	1.974	.494	2.74	.766	3.58	1.25	.312	−156.8	.987	1.753	−.002
7.77	1.96	.49	2.73	.77	3.55	1.25	.312	−56.8	.98	1.75	−.006
8.93	1.892	.473	2.70	.808	3.33	1.21	.302	−13.45	.946	1.754	−.023
9.57	1.882	.471	2.65	.798	3.34	1.22	.305	−12.20	.941	1.739	−.025

(1) parabolic area correction applied beyond $\epsilon = 6.[illegible]\%$ (3) $A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$

(2) $\Delta u = \bar{\sigma}_{ac} - \bar{\sigma}_a$

Soil Properties

Test No. $\overline{C(\frac{1}{K_o})UE}$-3 — Tested by J. V. 1/1964 — $\bar{\sigma}_{1c} = 6.00\ kg/cm^2$ $\bar{\sigma}_{3c} - 3.20\ kg/cm^2$

Batch No. S-6 — $d\varepsilon/dt = 0.000048$ inch/min — P. P. R. = 92%

2 spiral paper drains — back pressure = $3.00\ kg/cm^2$

$\Delta\sigma_r = 0$ and σ_a decreased — $t_c = 4$ days

$\bar{\sigma}_{1c}$ kg/cm^2	Volume cc	Length cm	Area cm^2	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	87.26	8.64	10.10	30.6	95.40	.892	—	1.919
2.00	80.06	—	—	25.4	95.84	.736	8.3	2.008
4.00	77.89	—	—	24.0	97.04	.688	10.8	2.042
6.00	75.32	8.86	8.50	22.85	100	.635	13.6	2.088

circ. at end of test : 3.314 in.

Stress-Strain Data

Test No. $\overline{C(\frac{1}{K_o})UE-3}$ Tested by J.V. 1/1964 $\bar{\sigma}_{1c} = 6.00\ kg/cm^2$ $\bar{\sigma}_{3c} = 3.20\ kg/cm^2$
$w_i = 30.60\ \%$ $w_f = 22.85\ \%$

(all stresses in kg/cm^2)

ϵ %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\bar{\sigma}_r / \bar{\sigma}_a$	Δu	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (3)	A	$\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$
0	2.80	.466	6.00	3.20	1.875	0	0	—	1.40	4.60
.02	3.455	.576	6.34	2.885	2.20	.315	.052	.481	1.727	4.612
.17	3.48	.581	5.56	2.08	2.68	1.12	.187	1.65	1.74	3.82
.42	14	.573	5.42	1.98	2.74	1.22	.203	1.91	1.72	3.70
.66	34	.556	5.08	1.74	2.92	1.46	.243	2.70	1.67	3.41
.92	3.30	.549	4.94	1.64	3.01	1.56	.260	3.12	1.65	3.29
1.07	3.285	.548	4.89	1.605	3.05	1.595	.266	3.29	1.642	3.247
1.26	3.25	.541	4.76	1.51	3.16	1.69	.282	3.76	1.625	3.135
1.48	3.24	.540	4.69	1.45	3.24	1.75	.292	3.72	1.62	3.07
1.83	3.24	.540	4.58	1.34	3.42	1.86	.310	4.23	1.62	2.96
1.98	3.21	.535	4.56	1.35	3.38	1.85	.308	4.51	1.605	2.955
3.24	3.16	.527	4.40	1.255	3.50	1.945	.324	5.40	1.58	2.835
3.30	3.10	.517	4.30	1.20	3.58	2.00	.333	6.67	1.55	2.75
3.58	3.095	.516	4.25	1.155	3.68	2.045	.341	6.94	1.547	2.702
3.77	3.075	.513	4.23	1.155	3.67	2.045	.341	7.43	1.537	2.692
3.95	3.06	.510	4.22	1.16	3.64	2.04	.340	7.85	1.53	2.69
4.74	2.985	.498	4.13	1.145	3.60	2.055	.342	11.10	1.492	2.637
5.18	2.926	.487	4.09	1.164	3.52	2.036	.339	16.15	1.463	2.627
(2) 5.46	2.87	.478	4.04	1.17	3.46	2.03	.338	+29.0	1.435	2.605
6.32	2.66	.443	3.87	1.21	3.20	1.99	.332	−14.2	1.33	2.54
6.79	2.61	.435	3.86	1.25	3.09	1.95	.325	−10.25	1.305	2.555
(1) 7.64	2.586	.431	3.76	1.174	3.20	2.026	.338	−9.46	1.293	2.467
8.54	2.463	.411	3.62	1.157	3.13	2.043	.341	−6.06	1.231	2.388
9.42	2.211	.368	3.49	1.279	2.73	1.92	.320	−3.26	1.105	2.384
10.90	1.886	.314	3.32	1.434	2.31	1.765	.294	−1.93	.943	2.377

(1) parabolic area corrections applied beyond $\epsilon = 7.64\ \%$
(2) necking started
(3) $\Delta u = \bar{\sigma}_{ac} - \bar{\sigma}_a$

Soil Properties

Test No. $C(\overline{\frac{1}{K_o})URC}-2$ Tested by J.V. 8/1963 $\bar{\sigma}_{1c}$: 4.00 kg/cm² $\bar{\sigma}_{3c}$ = 2.14 kg/cm²

Batch No. 5-5 stress controlled, 12 days to failure; P.P.R. not measured

without paper drains back pressure 3.00 kg/cm²

$\Delta\sigma_r = 0$ and σ_a increased t_c = 2 days

$\bar{\sigma}_{1c}$ kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	88.15	8.65	10.20	30.2	92.0	.91	—	1.90
2.00	80.64	—	—	27.2	100	.754	8.2	2.01
3.00	79.64	—	—	25.9	100	.72	10.0	2.03
4.00	77.71	8.80	8.82	25.0	100	.69	11.5	2.05

circ. at end of test: not measured

consolidated against back pressure

Stress-Strain Data

Test No. $\overline{C(\frac{1}{K_o})}URC-2$ Tested by J.V. 8/1963 $\bar{\sigma}_{1c} = 4.00\ kg/cm^2$ $\bar{\sigma}_{3c} = 2.14\ kg/cm^2$

$w_i = 30.2\ \%$ $w_f = 25.0\ \%$

(all stresses in kg/cm^2)

ϵ %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_a$	$\bar{\sigma}_r$	$\bar{\sigma}_a / \bar{\sigma}_r$	$\Delta u^{(2)}$	$\frac{\Delta u}{\bar{\sigma}_{1c}}$	$A^{(3)}$	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$	$\Delta\sigma_a - \Delta\sigma_r$
0	1.86	.465	2.14	4.00	.535	0	0	—	.93	3.07	0
.058	1.00	.250	2.59	3.59	.722	.41	.10	.478	.50	3.09	.86
.086	.66	.14	2.67	3.33	.802	.67	.17	.556	.33	3.00	1.20
.16	.33	.082	2.53	2.86	.885	1.14	.28	.746	.165	2.695	1.53
.17	0	0	2.73	2.73	1.00	1.27	.33	.685	0	2.730	1.86
.25	.298	.077	2.678	2.38	1.125	1.62	.41	.75	.149	2.529	2.158
.376	.598	.149	2.608	2.01	1.29	1.99	.497	.778	.299	2.309	2.458
.491	.902	.225	2.682	1.78	1.51	2.22	.555	.804	.451	2.231	2.762
.685	1.108	.277	2.638	1.53	1.72	2.47	.617	.832	.554	2.084	2.968
.94	1.30	.325	2.73	1.43	1.91	2.57	.642	.813	.65	2.08	3.16
1.38	1.50	.375	2.70	1.20	2.25	2.80	.70	.833	.75	1.95	3.36
1.70	1.605	.401	2.695	1.09	2.47	2.91	.727	.840	.802	1.892	3.465
1.77	1.662	.413	2.712	1.05	2.58	2.95	.737	.836	.831	1.881	3.522
2.03	1.702	.423	2.722	1.02	2.67	2.98	.745	.837	.851	1.871	3.562
2.11	1.752	.438	2.802	1.05	2.67	2.95	.762	.817	.876	1.926	3.612
2.20	1.796	.449	2.816	1.02	2.76	2.98	.745	.815	.898	1.918	3.656
2.38	1.911	.477	2.951	1.04	2.84	2.96	.74	.785	.955	1.995	3.771
2.47	2.01	.502	3.05	1.04	2.93	2.96	.74	.765	1.005	2.045	3.87
3.46	2.109	.502	3.069	.96	3.20	3.04	.76	.765	1.054	2.014	3.969
5.88	2.178	.544	3.038	.66	3.53	3.14	.785	.778	1.089	1.949	4.038
(1) 9.76	2.191	.547	3.021	.83	3.64	3.17	.792	.783	1.095	1.925	4.051
21.00	2.186	.546	2.926	.74	3.96	3.26	.815	.805	1.093	1.833	4.046

(1) parabolic area correction applied beyond $\epsilon = 9.76\ \%$

(1) last load increase applied

(2) $\Delta u = \bar{\sigma}_{rc} - \bar{\sigma}_r$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$

Soil Properties

Test No. $\overline{C(\frac{1}{K_o})URC-4}$ Tested by J.V. 9/1963 $\bar{\sigma}_{1c}$ = 4.00 kg/cm² $\bar{\sigma}_{3c}$ = 2.13 kg/cm²

Batch No. S-5   Stress controlled, 10 days to failure; P.P.R. = 100 %

8 vertical overlapping paper drains   back pressure 3.00 kg/cm²

$\Delta\sigma_r = 0$ and σ_a increased t_c = 3 days

$\bar{\sigma}_{1c}$ kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	87.00	8.34	10.15	30.9	94.5	.91	—	1.90
2.00	83.18	—	—	28.2	94.8	.826	4.4	1.95
3.00	80.44	—	—	26.4	98.6	.768	7.4	1.99
4.00	76.78	8.76	8.87	24.9	100	.689	11.6	2.05

circ. at end of test: 5.029 in.

Stress–Strain Data

Test No. $\overline{C(\frac{1}{K_o})}URC-4$ Tested by J.V. 9/1963 $\bar{\sigma}_{1c} = 4.00\ kg/cm^2$ $\bar{\sigma}_{3c} = 2.13\ kg/cm^2$

$w_i = 30.9\%$ $w_f = 24.9\%$

(all stresses in kg/cm^2)

ε %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_a$	$\bar{\sigma}_r$	$\bar{\sigma}_a / \bar{\sigma}_r$	Δu	$\frac{\Delta u}{\bar{\sigma}_{1c}}$	A	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$	$\frac{\Delta\sigma_a - \Delta\sigma_r}{\bar{\sigma}_{1c}}$
0	1.87	.467	2.13	4.00	.533	0	0	—	.935	3.065	0
.001	1.42	.33	2.46	3.88	.635	.12	.03	.266	.71	3.17	.137
.05	1.01	.252	2.39	3.40	.70	.60	.15	.696	.505	2.895	.215
.07	.67	.167	2.57	3.24	.79	.76	.19	.632	.335	2.905	.300
.14	.34	.085	2.63	2.97	.89	1.03	.26	.674	.17	2.80	.382
.21	.01	.002	2.64	2.65	.996	1.35	.34	.725	.005	2.645	.465
.28	.296	.074	2.686	2.39	1.12	1.62	.41	.746	.148	2.538	.541
.42	.588	.147	2.708	2.12	1.27	1.88	.47	.765	.294	2.414	.614
.64	.882	.22	2.742	1.86	1.47	2.14	.54	.778	.442	2.302	.687
.90	1.18	.295	2.78	1.60	1.74	2.40	.60	.787	.59	2.19	.762
1.68	1.43	.357	2.71	1.28	2.12	2.72	.68	.824	.715	1.995	.824
2.31	1.62	.405	2.76	1.14	2.42	2.86	.72	.819	.81	1.95	.872
3.48	1.815	.454	2.825	1.01	2.80	2.99	.75	.79	.907	1.917	.921
4.80	1.94	.435	2.86	.92	3.11	3.08	.77	.808	.97	1.89	.952
(1) 6.10	2.018	.504	2.908	.89	3.27	3.11	.777	.801	1.009	1.899	.972
6.42	2.07	.517	2.97	.90	3.30	3.10	.775	.787	1.035	1.935	.985
8.39	2.11	.527	2.99	.88	3.40	3.12	.78	.784	1.055	1.935	.995
11.35	2.11	.527	3.00	.89	3.37	3.11	.777	.781	1.055	1.945	.995
15.30	2.00	.50	2.84	.84	3.38	3.16	.79	.816	1.00	1.84	.967
18.20	1.938	.484	2.738	.80	3.43	3.20	.80	.839	.969	1.769	.952
(2) 19.00	1.92	.48	2.74	.82	3.34	3.18	.795	.84	.96	1.78	.948

(1) parabolic area correction applied beyond $\varepsilon = 6.10\%$

(2) last load increase applied

Soil Properties

Test No. $\overline{C(\frac{1}{K_o})URC-5}$ Tested by J.V. 12/1963 $\bar{\sigma}_{1c} = 4.00$ kg/cm² $\bar{\sigma}_{3c} = 1.988$ kg/cm²

Batch No. S-6 $d\epsilon/dt = 0.00012$ inch/min P.P.R. = 100 %

8 vertical overlapping paper drains back pressure 3.00 kg/cm²

$\Delta\sigma_r = 0$ and σ_a increased $t_c = 4$ days

$\bar{\sigma}_{1c}$ kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	86.40	8.59	10.05	29.7	97.1	.846	—	1.95
1.50	82.19	—	—	26.4	97.3	.756	4.9	2.01
3.00	78.49	—	—	23.7	97.6	.678	9.1	2.06
4.00	76.77	8.76	8.76	22.8	100	.637	11.3	2.09

circ. at end of test: 4.660 in

Stress - Strain Data

Test No. $\overline{C(\frac{1}{K_o})}$URC-5 Tested by J.V. 12/1963 $\bar{\sigma}_{1c} = 4.00$ kg/cm² $\bar{\sigma}_{3c} = 1.988$ kg/cm²
$w_i = 29.7$ % $w_f = 22.8$ %

(all stresses in kg/cm²)

ϵ %	$\sigma_1 - \sigma_3$ corrected	$\frac{\sigma_1-\sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\bar{\sigma}_a/\bar{\sigma}_r$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	$\frac{\bar{\sigma}_1-\bar{\sigma}_3}{2}$	$\frac{\bar{\sigma}_1+\bar{\sigma}_3}{2}$	$\frac{\Delta\bar{\sigma}_a-\Delta\bar{\sigma}_r}{\bar{\sigma}_{1c}}$
0	2.012	.503	4.00	1.988	.497	0	0	—	1.006	2.994	0
.014	1.777	.444	3.99	2.213	.554	.01	.002	.043	.888	3.101	.059
.028	1.262	.316	3.70	2.438	.658	.30	.075	.40	.631	3.069	.187
.095	.474	.118	3.16	2.686	.85	.84	.21	.546	.237	2.923	.385
.37	.638	.159	2.24	2.878	1.28	1.76	.44	.665	.319	2.559	.662
.61	1.03	.258	1.90	2.93	1.54	2.10	.525	.691	.515	2.415	.761
1.205	1.291	.322	1.54	2.831	1.84	2.46	.615	.744	.645	2.185	.825
2.76	1.876	.469	1.02	2.896	2.84	2.98	.745	.766	.938	1.958	.972
3.01	2.056	.513	.99	3.046	3.08	3.01	.753	.74	1.028	2.018	1.016
3.20	2.075	.519	.98	3.055	3.12	3.02	.755	.74	1.037	2.017	1.022
3.81	2.207	.551	.95	3.157	3.32	3.05	.762	.723	1.103	2.053	1.054
4.02	2.232	.558	.92	3.152	3.42	3.08	.77	.726	1.116	2.036	1.061
4.93	2.41	.603	.90	3.31	3.68	3.10	.775	.701	1.205	2.105	1.106
(1) 7.60	2.48	.62	.88	3.36	3.82	3.12	.78	.695	1.24	2.12	1.123
8.57	2.515	.629	.88	3.395	3.84	3.12	.78	.689	1.257	2.137	1.132
9.36	2.53	.633	.89	3.41	3.88	3.12	.78	.687	1.265	2.145	1.136
10.10	2.54	.635	.89	3.43	3.86	3.11	.778	.683	1.27	2.16	1.138
10.30	2.569	.642	.90	3.469	3.86	3.10	.775	.677	1.284	2.184	1.145
11.10	2.57	.643	.91	3.48	3.82	3.09	.772	.674	1.285	2.195	1.146
13.72	2.654	.664	.92	3.574	3.88	3.08	.77	.661	1.327	2.247	1.167
15.73	2.485	.621	.91	3.395	3.73	3.09	.772	.688	1.242	2.152	1.124

(1) parabolic area correction applied beyond $\epsilon = 7.60$ %
(2) $\Delta u = \bar{\sigma}_{rc} - \bar{\sigma}_r$
(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$

Soil Properties

Test No. $\overline{C(\frac{1}{K_o})URC}-6$ Tested by J. V.   1/1964 $\bar{\sigma}_{1c} = 6.00\ kg/cm^2$ $\bar{\sigma}_{3c} = 2.90\ kg/cm^2$

Batch No. 5-6 $d\epsilon/dt$ = 0.00012 inch/min   P. P. R. = 98 %

8 vertical overlapping paper drains   back pressure 3.00 kg/cm^2

$\Delta\sigma_r = 0$ and $\Delta\sigma_a$ increased t_c = 2 days

$\bar{\sigma}_{1c}$ kg/cm²	Volume cc	Length cm	Area cm²	w %	S %	e	$\frac{\Delta e}{1+e_o}$ %	γ_t g/cc
Initial	87.60	8.63	10.15	31.1	93.7	.920	—	1.89
2.00	79.53	—	—	26.1	97.3	.745	9.1	2.00
4.00	76.06	—	—	23.3	98.5	.667	13.2	2.06
6.00	73.95	8.92	8.29	22.4	100	.625	15.3	2.09

circ. at end of test : 4.492 in.

Stress - Strain Data

Test No. $C(\overline{\frac{1}{K_0}})$URC-6 Tested by J. V. 1/1964 $\bar{\sigma}_{1c}$ = 6.00 kg/cm² $\bar{\sigma}_{3c}$ = 2.90 kg/cm²
w_i = 31.1 % w_f = 22.4 %

(all stresses in kg/cm²)

ϵ %	$\sigma_1 - \sigma_a$ corrected	$\frac{\sigma_1-\sigma_3}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\bar{\sigma}_a/\bar{\sigma}_r$	Δu	$\frac{\Delta u}{\bar{\sigma}_{1c}}$	A	$\frac{\sigma_1-\sigma_3}{2}$	$\frac{\bar{\sigma}_1+\bar{\sigma}_3}{2}$	$\Delta(\bar{\sigma}_1-\bar{\sigma}_r)$
0	3.10	.517	6.00	2.90	.483	0	0	—	1.55	4.45	0
.03	2.06	.343	5.41	3.35	.619	.59	.098	.57	1.03	4.38	1.04
.09	1.382	.231	4.93	3.548	.72	1.07	.178	.625	.691	4.239	1.718
.154	1.071	.179	4.45	3.379	.758	1.45	.242	.715	.535	3.914	2.029
.43	.47	.078	3.30	3.770	1.142	2.70	.498	.76	.235	3.535	3.570
1.007	1.49	.248	2.26	3.75	1.458	3.74	.623	.82	.745	3.005	4.590
1.46	1.851	.308	1.92	3.771	1.96	4.08	.680	.82	.925	2.845	4.951
4.10	2.793	.465	1.34	4.133	3.08	4.66	.777	.795	1.396	2.736	5.89
4.70	2.905	.483	1.22	4.125	3.31	4.78	.797	.795	1.452	2.672	6.005
5.01	3.002	.502	1.21	4.212	3.48	4.79	.799	.785	1.501	2.711	6.10
5.72	3.141	.524	1.20	4.391	3.60	4.80	.800	.77	1.570	2.77	6.24
6.04	3.182	.530	1.20	4.382	3.66	4.80	.800	.765	1.591	2.791	6.28
6.79	3.290	.549	1.15	4.440	3.86	4.85	.808	.76	1.645	2.795	6.39
(1) 7.23	3.431	.572	1.15	4.581	3.98	4.85	.808	.745	1.715	2.865	6.53
7.86	3.586	.598	1.15	4.736	4.11	4.85	.808	.725	1.793	2.943	6.69
9.13	3.826	.638	1.18	5.006	4.23	4.82	.804	.70	1.913	3.093	6.93
10.43	3.950	.658	1.16	5.110	4.40	4.84	.806	.685	1.975	3.135	7.05
11.35	4.080	.680	1.17	5.250	4.48	4.83	.805	.675	2.040	3.210	7.18
11.92	4.144	.691	1.18	5.32	4.50	4.82	.804	.66	2.07	3.25	7.34

(1) parabolic area corrections applied beyond ϵ = 7.23 %

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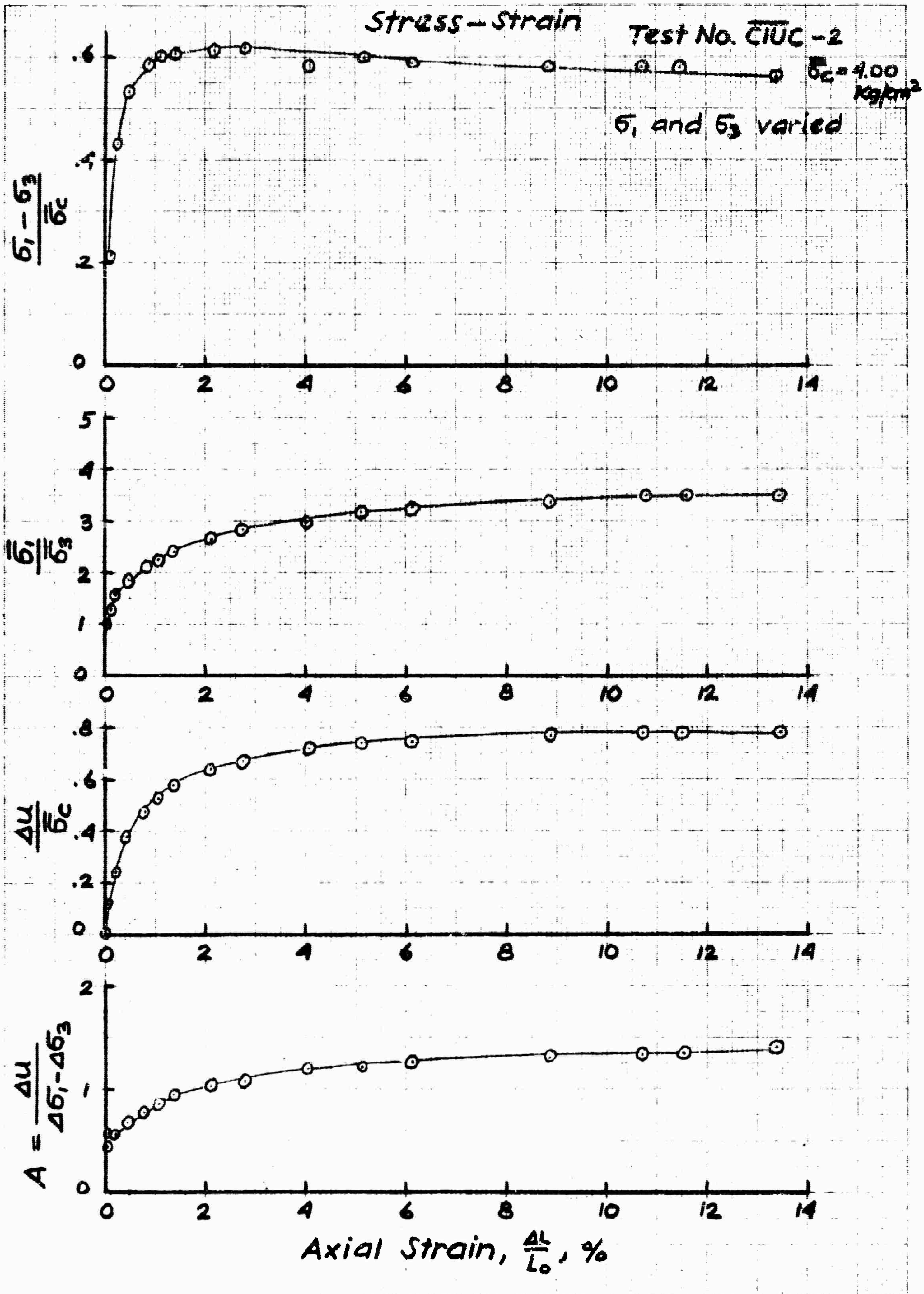
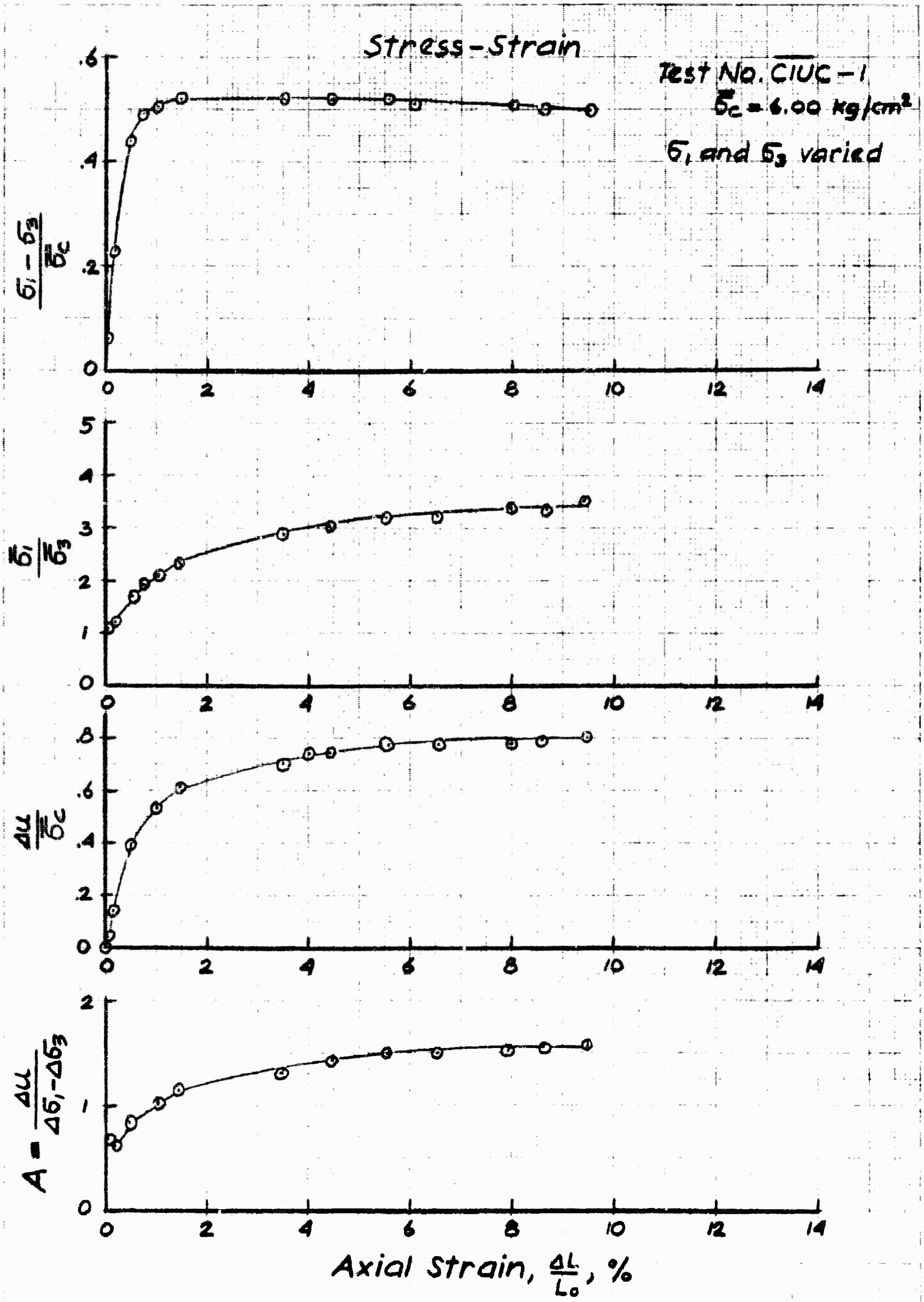
APPENDIX C

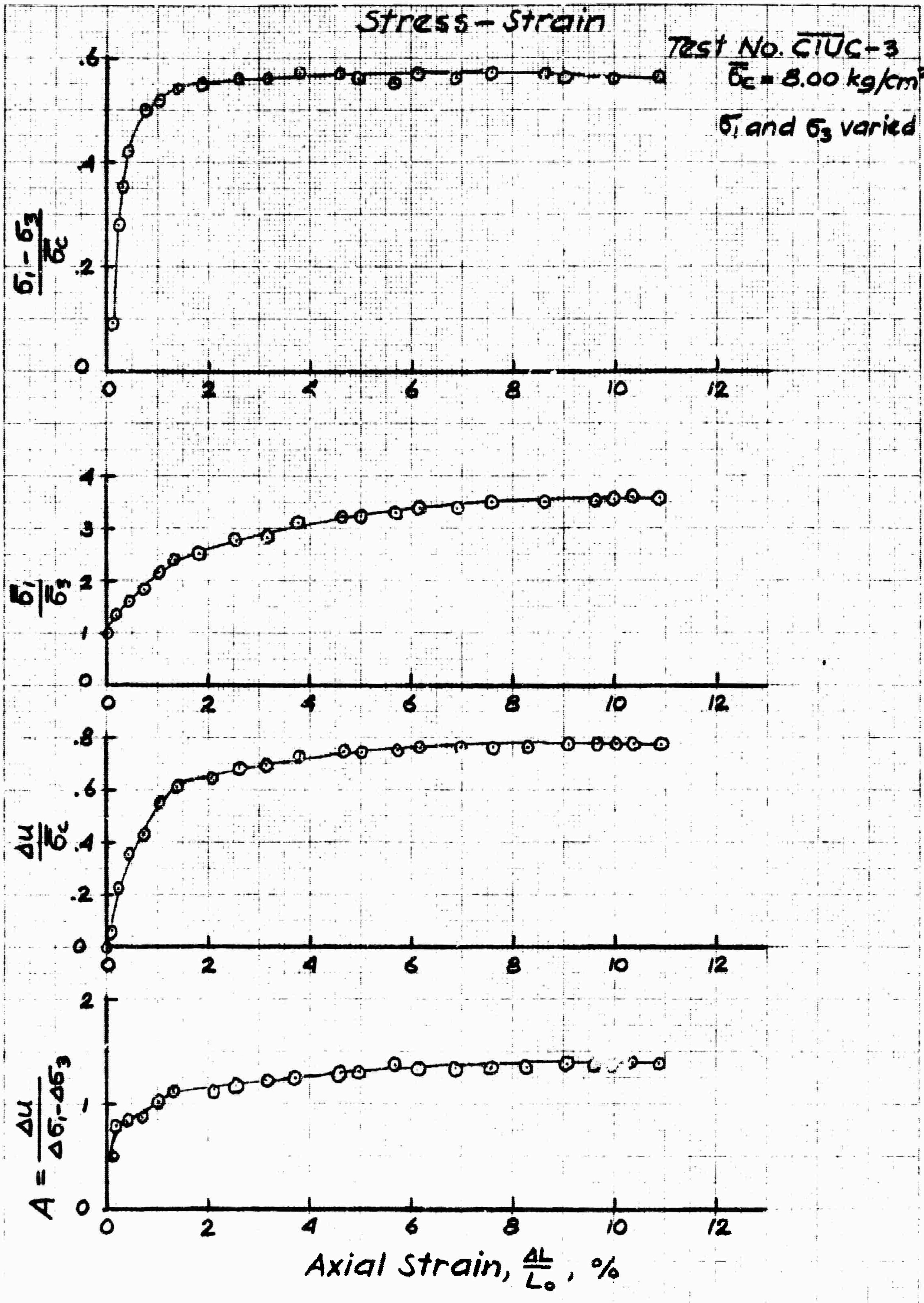
STRESS-STRAIN CURVES FROM $\overline{CU}$ TRIAXIAL TESTS ON NORMALLY CONSOLIDATED BOSTON BLUE CLAY PREPARED FROM A DILUTE SLURRY

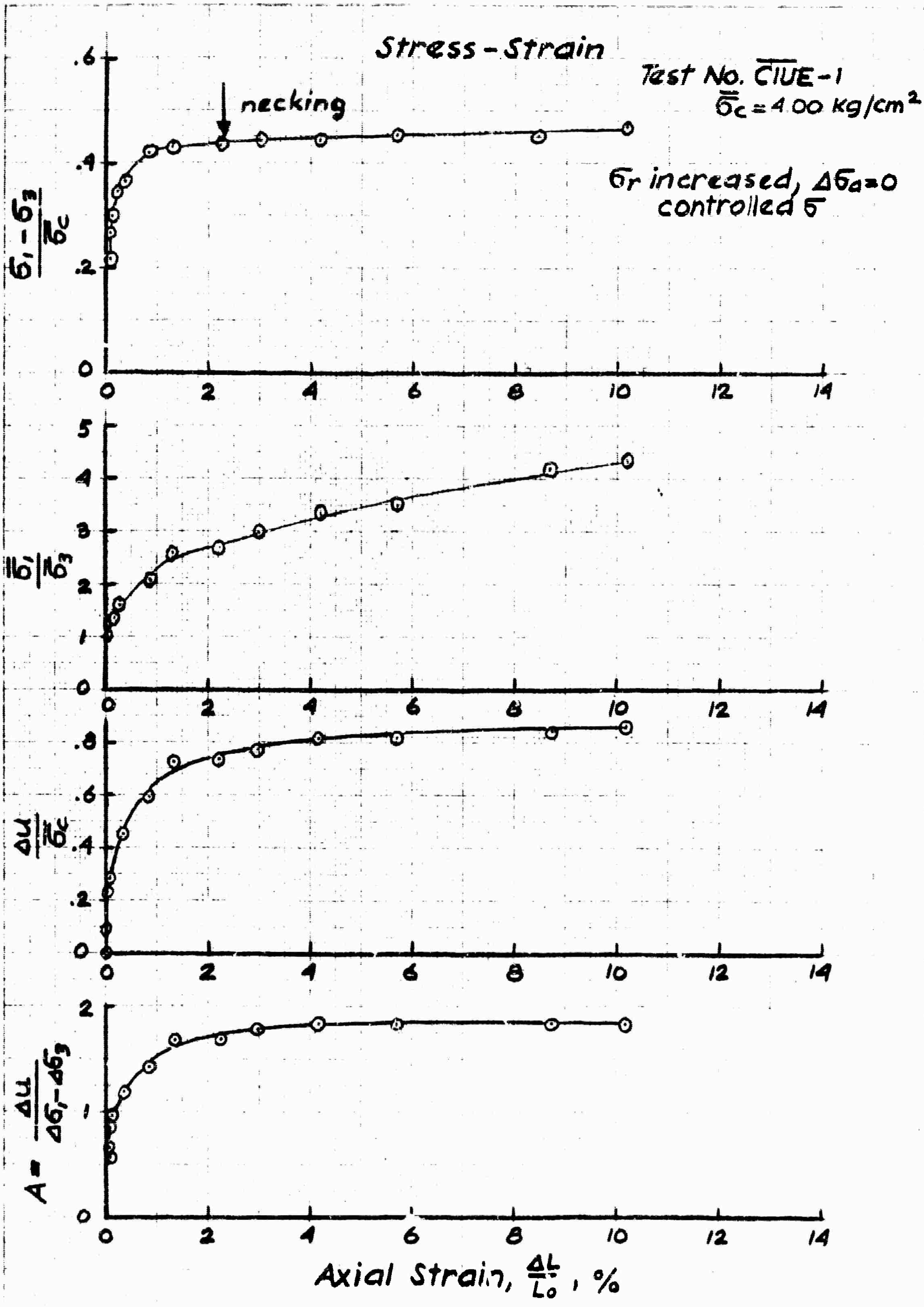


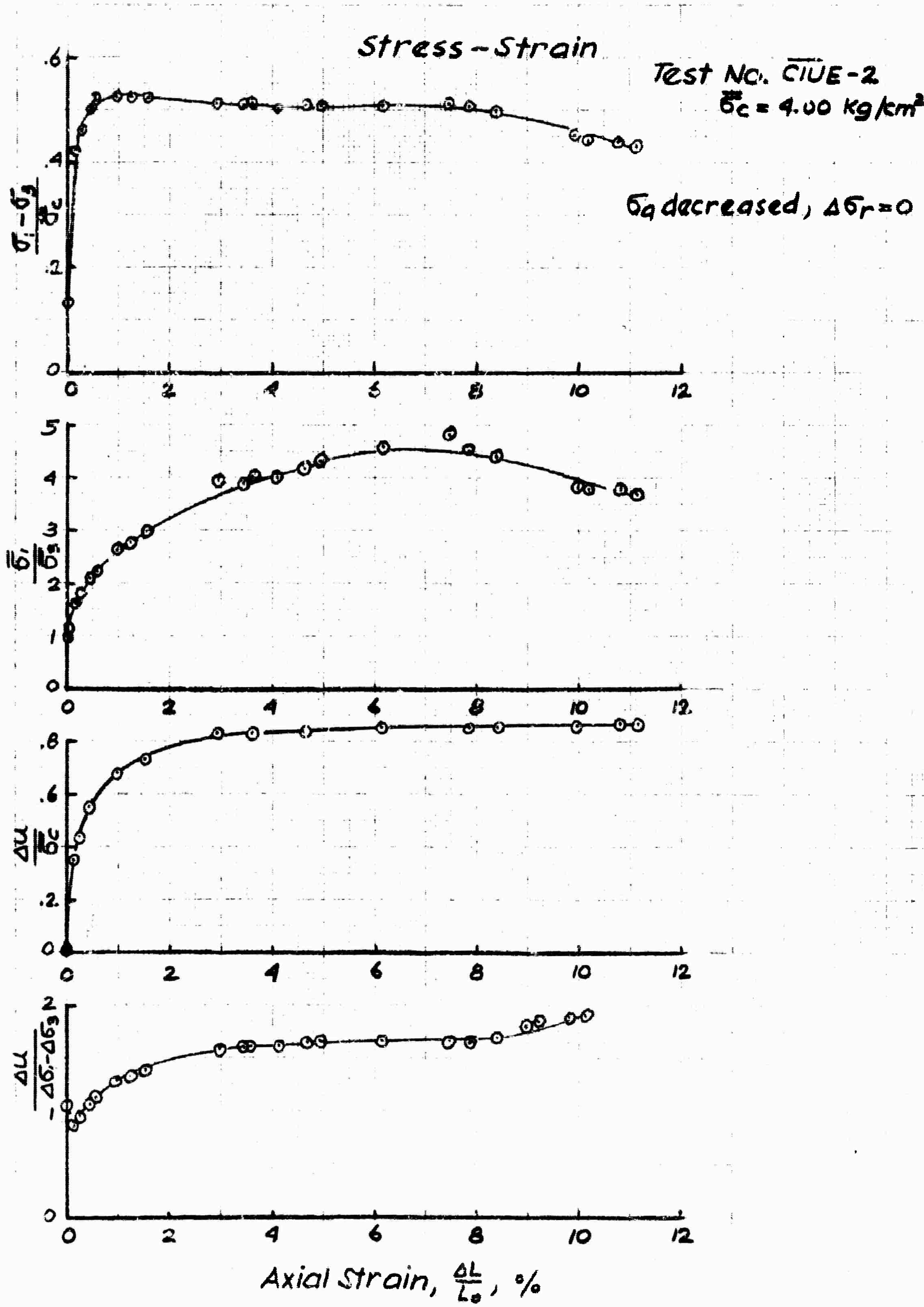
Stress-Strain
Test No. $\overline{CIUC}$-2
$\bar{\sigma}_c$ = 4.00 Kg/cm²
σ_1 and σ_3 varied
$\frac{\sigma_1-\sigma_3}{\bar{\sigma}_c}$
$\bar{\sigma}_1/\bar{\sigma}_3$
$\frac{\Delta u}{\bar{\sigma}_c}$
$A = \frac{\Delta u}{\Delta\sigma_1-\Delta\sigma_3}$
Axial Strain, $\frac{\Delta L}{L_o}$, %

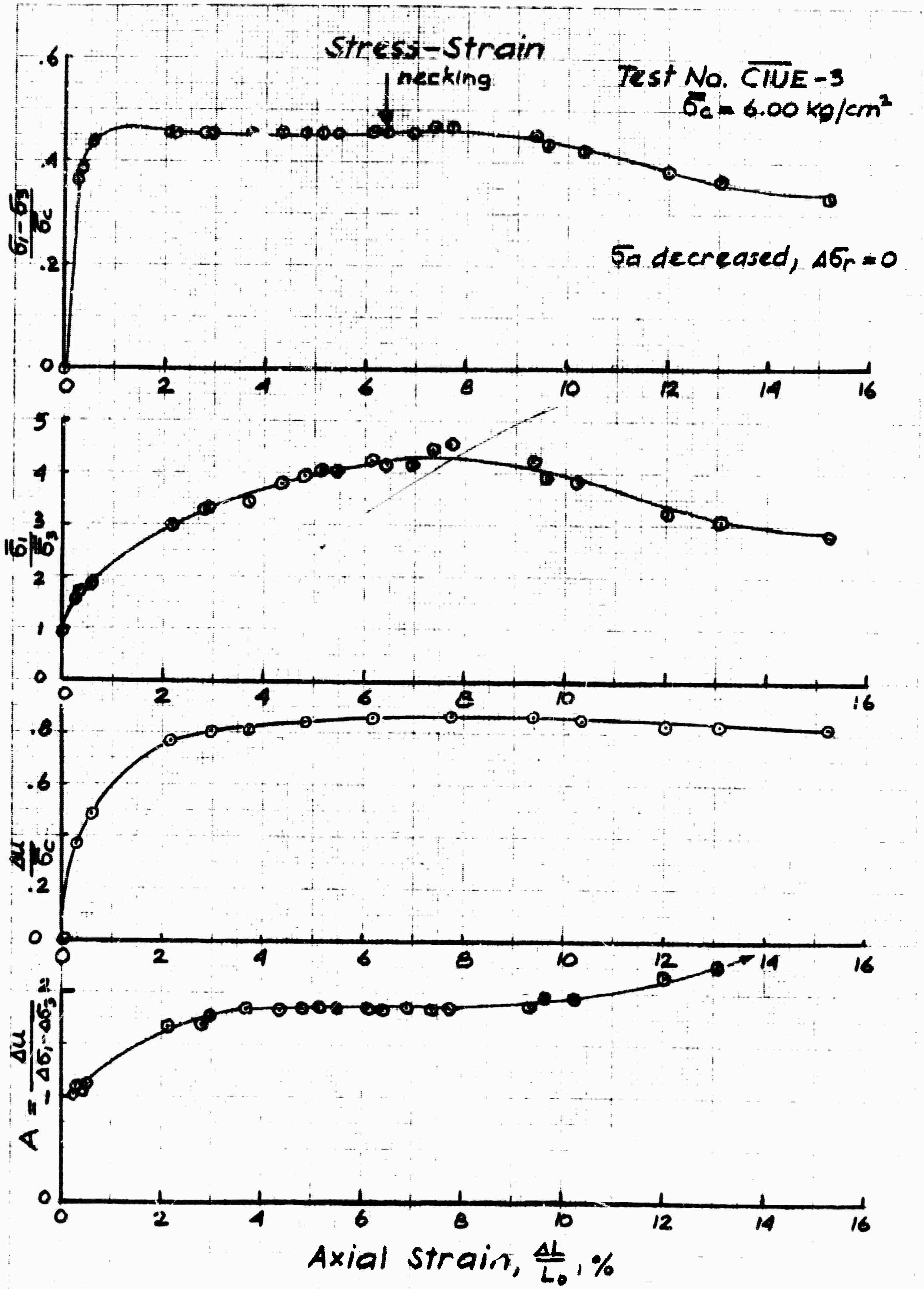


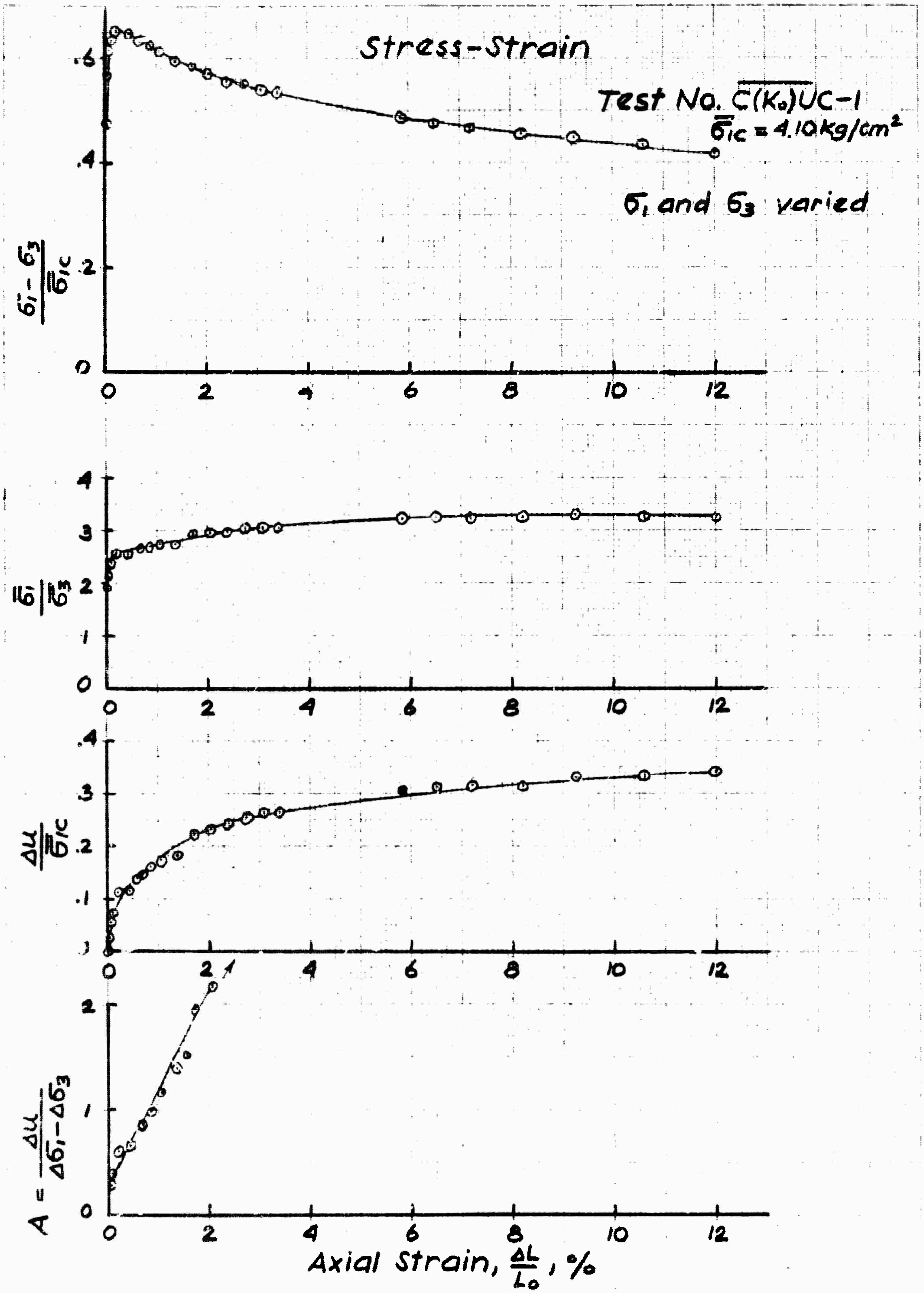
Stress-Strain
Test No. $\overline{CIUC}$-1
$\bar{\sigma}_c$ = 6.00 kg/cm²
σ_1 and σ_3 varied
$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_c}$
$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$
$\frac{\Delta u}{\bar{\sigma}_c}$
$A = \frac{\Delta u}{\Delta\sigma_1 - \Delta\sigma_3}$
Axial Strain, $\frac{\Delta L}{L_0}$, %


Stress – Strain
Test No. $\overline{CIUC}$-3
$\bar{\sigma}_c$ = 8.00 kg/cm²
σ_1 and σ_3 varied
$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_c}$
$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$
$\frac{\Delta u}{\bar{\sigma}_c}$
$A = \frac{\Delta u}{\Delta\sigma_1 - \Delta\sigma_3}$
Axial Strain, $\frac{\Delta L}{L_o}$, %

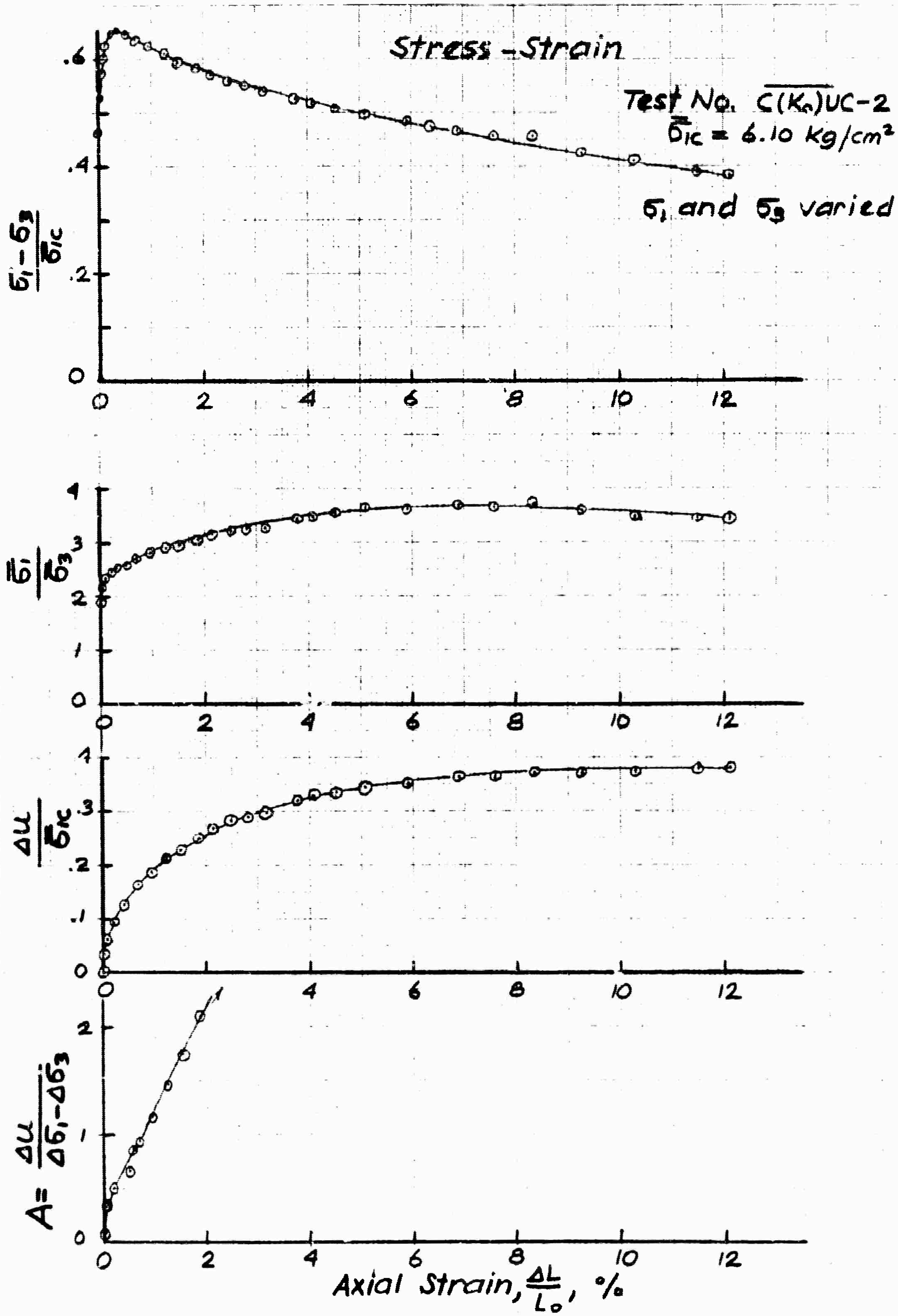

Stress - Strain
Test No. $\overline{CIUE}$-1
$\bar{\sigma}_c$ = 4.00 kg/cm²
σ_r increased, $\Delta\sigma_a$=0
controlled $\bar{\sigma}$
necking
$\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_c}$
$\bar{\sigma}_1 / \bar{\sigma}_3$
$\frac{\Delta u}{\bar{\sigma}_c}$
A = $\frac{\Delta u}{\Delta\sigma_1 - \Delta\sigma_3}$
Axial Strain, $\frac{\Delta L}{L_o}$, %


Stress-Strain
Test No. $\overline{CIUE}$-2
$\bar{\sigma}_c$ = 4.00 kg/cm²
σ_a decreased, $\Delta\sigma_r$=0
$\frac{\sigma_1-\sigma_3}{\bar{\sigma}_c}$
$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$
$\frac{\Delta u}{\bar{\sigma}_c}$
$\frac{\Delta u}{\Delta\sigma_1-\Delta\sigma_3}$
Axial Strain, $\frac{\Delta L}{L_o}$, %

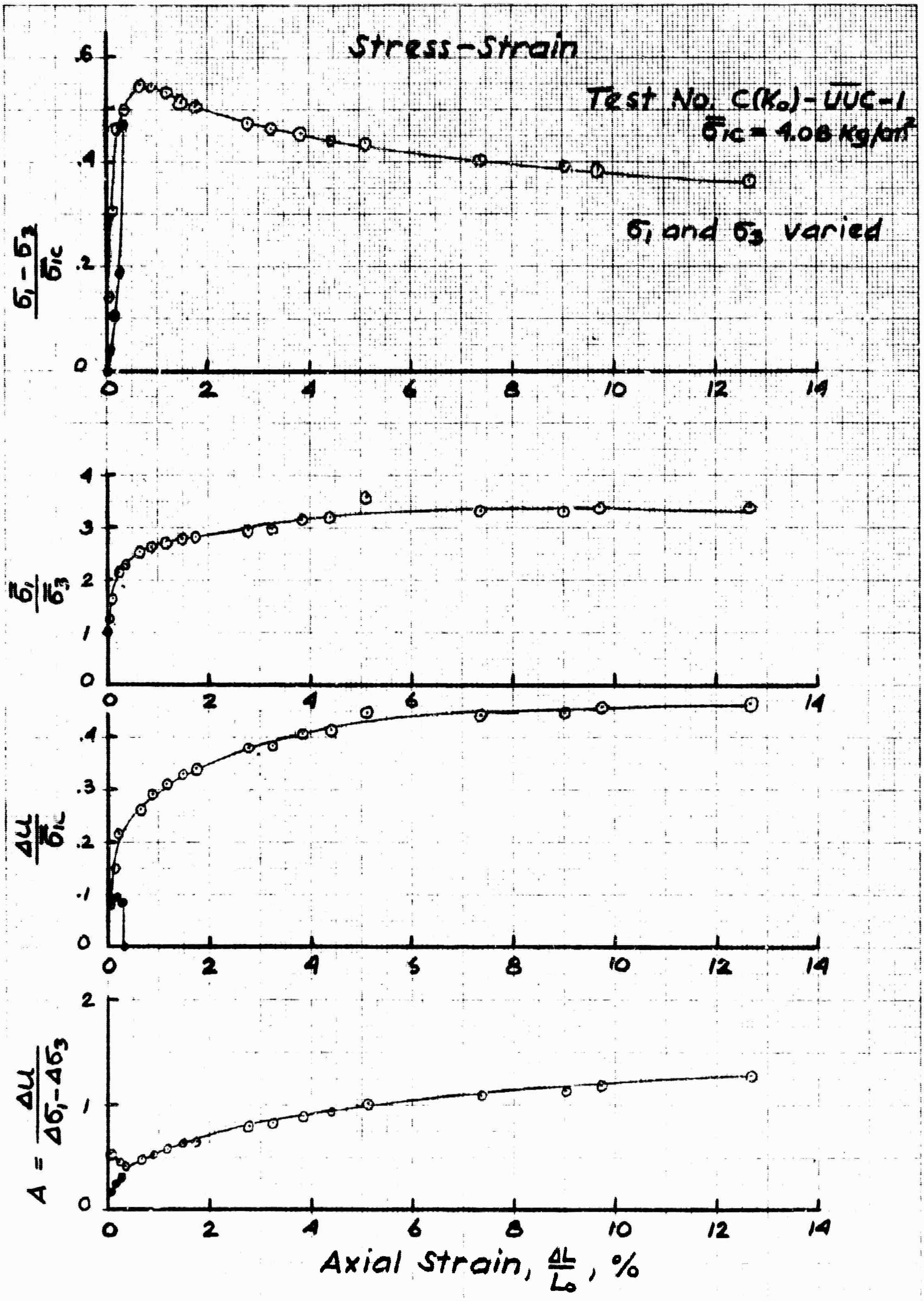

Stress-Strain
necking
Test No. $\overline{CIUE}$-3
$\bar{\sigma}_c$ = 6.00 kg/cm²
σ_a decreased, $\Delta\sigma_r = 0$
$\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_c}$
$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$
$\frac{\Delta u}{\bar{\sigma}_c}$
$A = \frac{\Delta u}{\Delta\sigma_1 - \Delta\sigma_3}$
Axial Strain, $\frac{\Delta L}{L_o}$, %

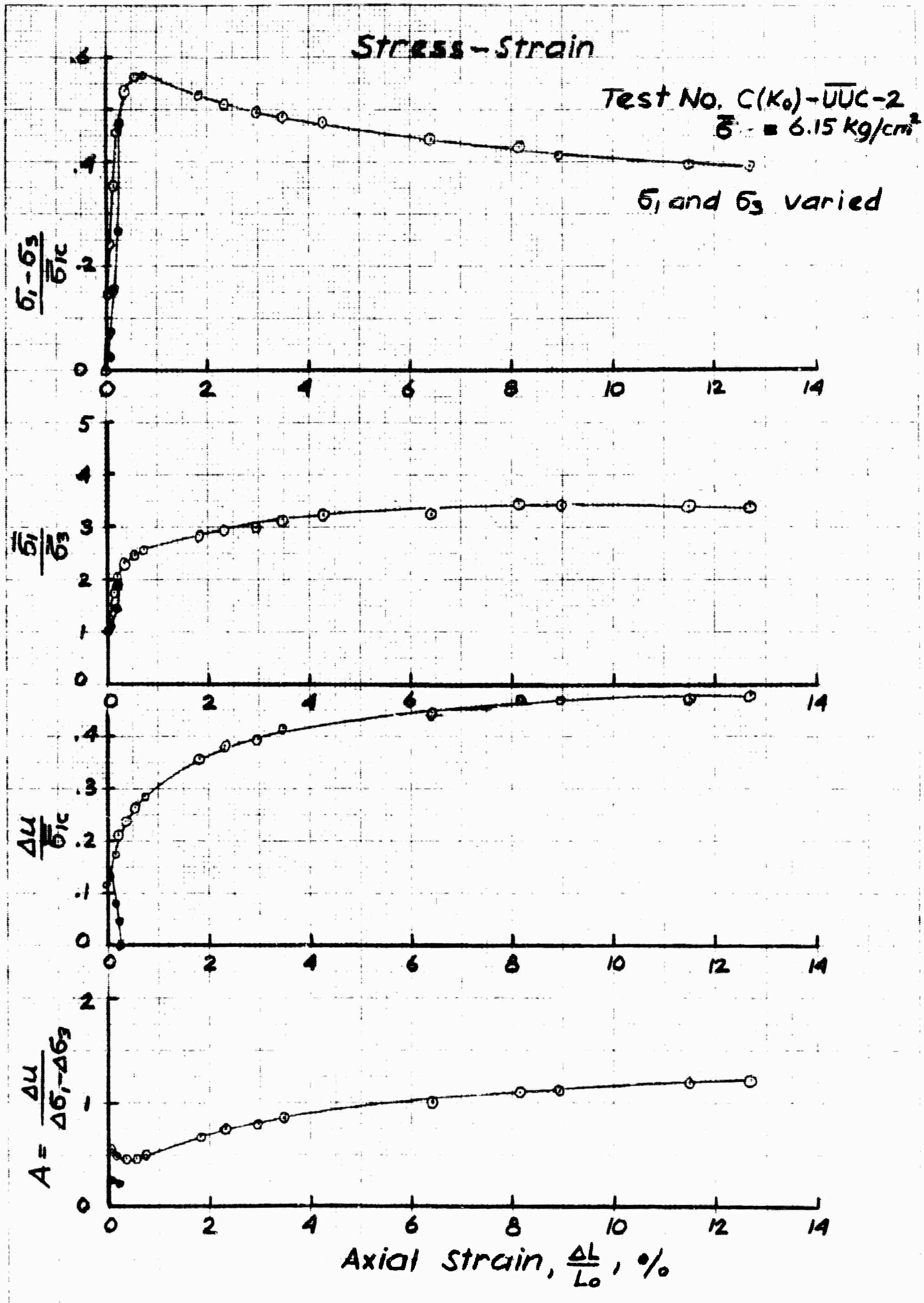


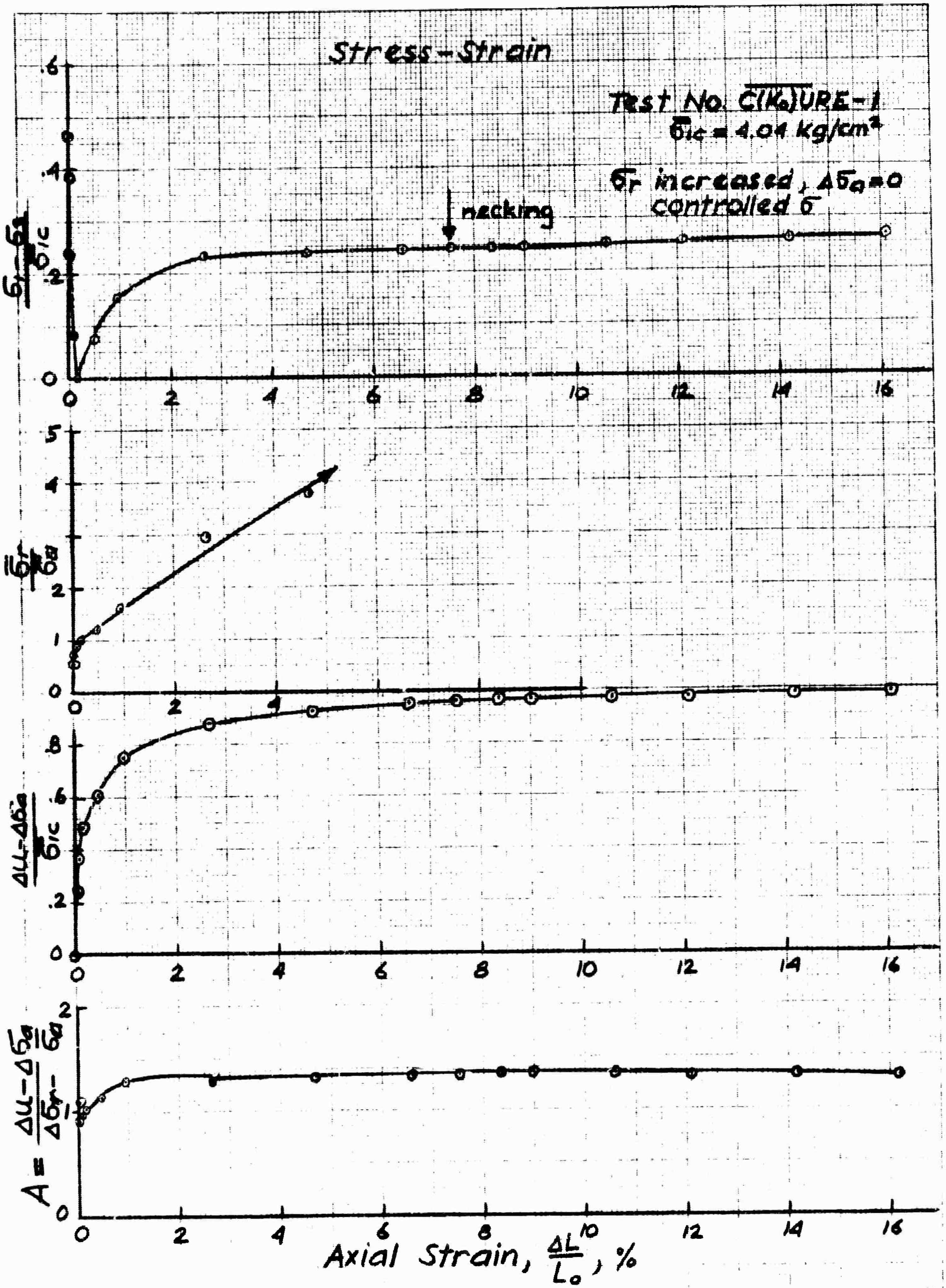
Stress-Strain
Test No. $\overline{C(K_o)UC}$-1
$\bar{\sigma}_{1c} = 4.10\ kg/cm^2$
σ_1 and σ_3 varied
$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$
$\bar{\sigma}_1 / \bar{\sigma}_3$
$\frac{\Delta u}{\bar{\sigma}_{1c}}$
$A = \frac{\Delta u}{\Delta\sigma_1 - \Delta\sigma_3}$
Axial Strain, $\frac{\Delta L}{L_o}$, %



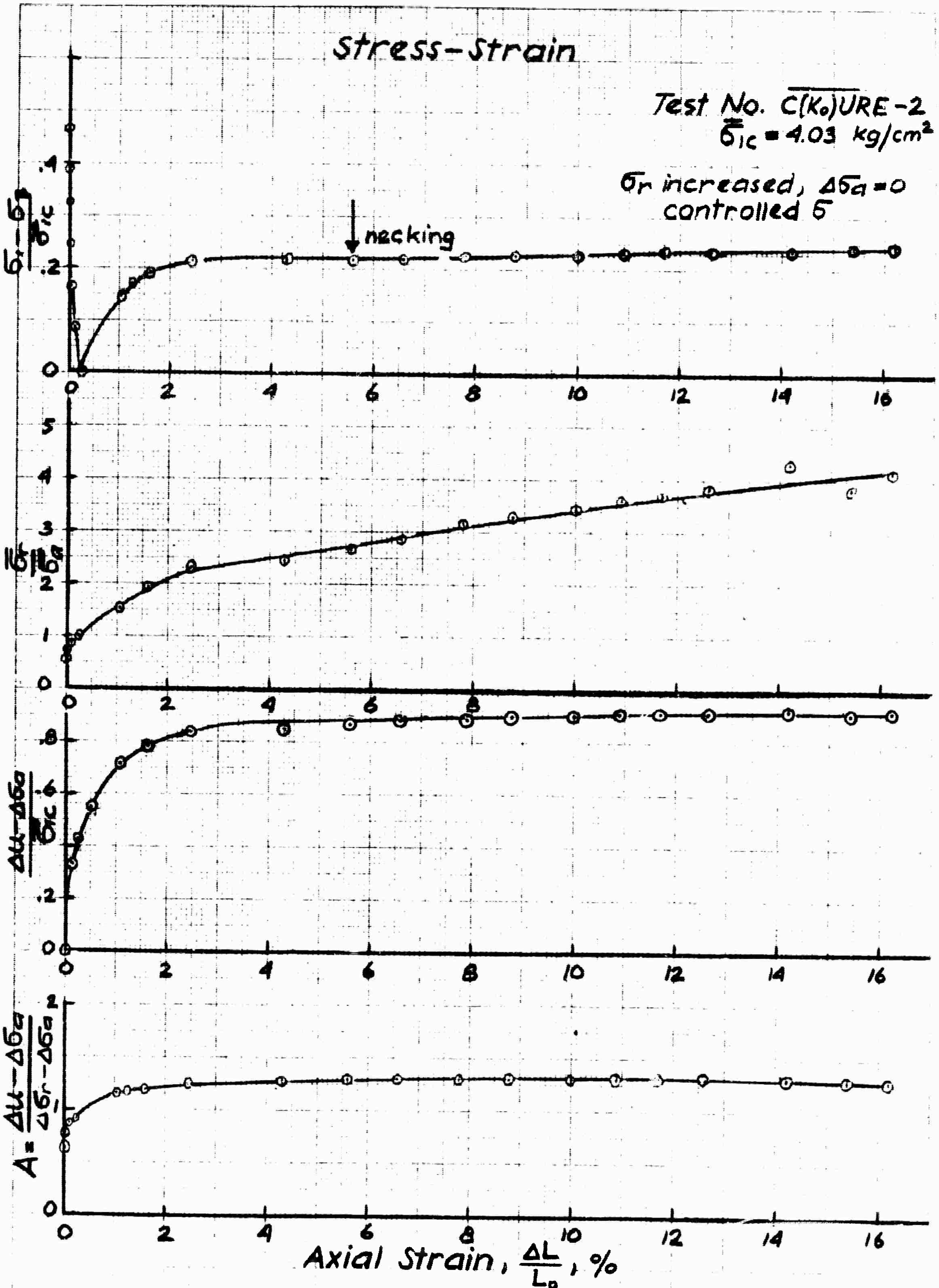
Stress - Strain
Test No. $\overline{C(K_o)UC-2}$
$\bar{\sigma}_{1c}$ = 6.10 kg/cm²
σ_1 and σ_3 varied
$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$
$\bar{\sigma}_1 / \bar{\sigma}_3$
$\frac{\Delta u}{\bar{\sigma}_{1c}}$
$A = \frac{\Delta u}{\Delta\sigma_1 - \Delta\sigma_3}$
Axial Strain, $\frac{\Delta L}{L_o}$, %


Stress-Strain
Test No. C(K₀)-UUC-1
$\bar{\sigma}_{1c}$ = 4.08 Kg/cm²
σ_1 and σ_3 varied
$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$
$\bar{\sigma}_1 / \bar{\sigma}_3$
$\frac{\Delta u}{\bar{\sigma}_{1c}}$
$A = \frac{\Delta u}{\Delta\sigma_1 - \Delta\sigma_3}$
Axial Strain, $\frac{\Delta L}{L_0}$, %

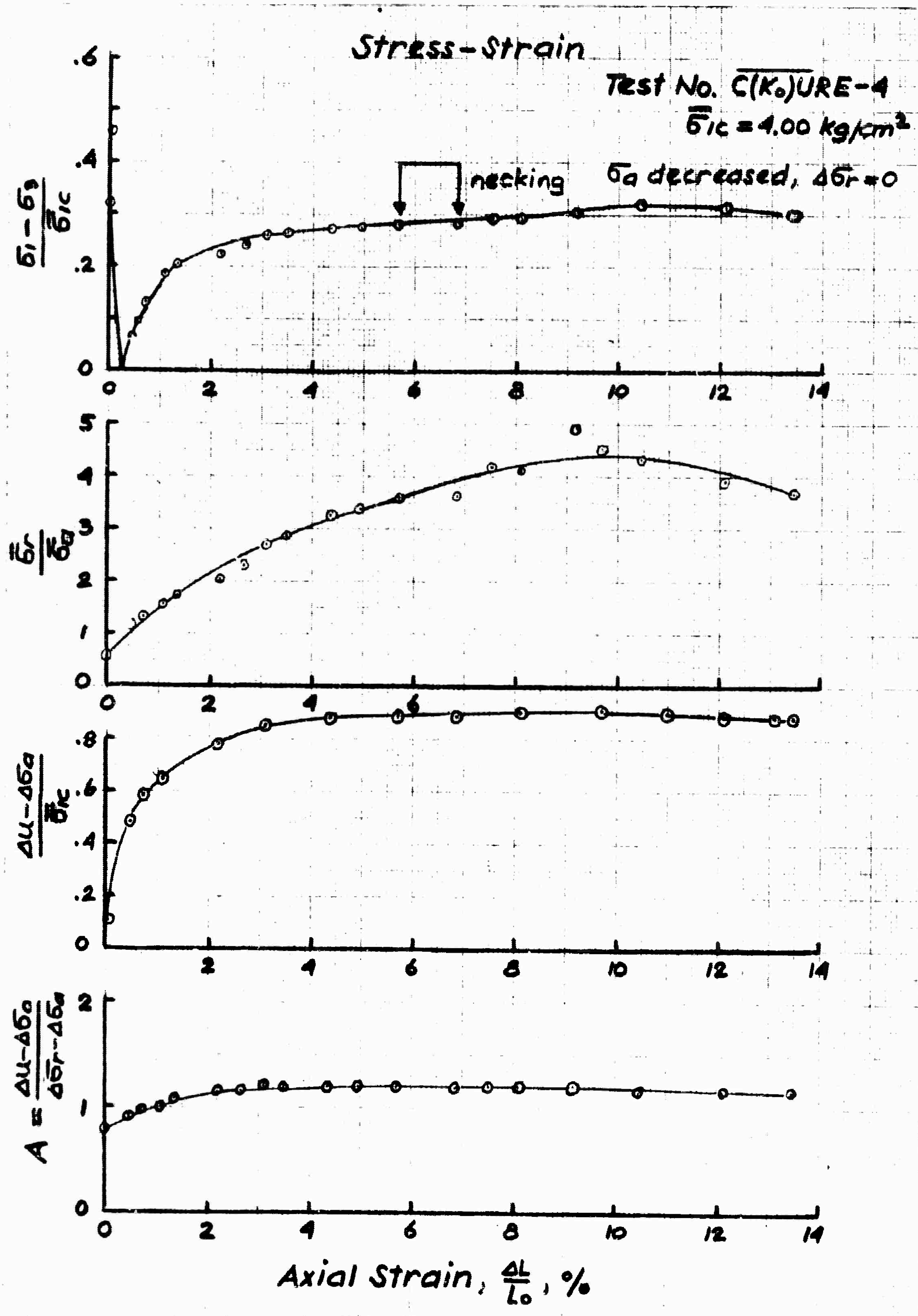

Stress-Strain
Test No. C(K₀)-ŪUC-2
σ̄ = 6.15 kg/cm²
σ₁ and σ₃ varied
(σ₁-σ₃)/σ̄₁c
σ̄₁/σ̄₃
Δu/σ̄₁c
A = Δu/(Δσ₁-Δσ₃)
Axial Strain, ΔL/Lo, %

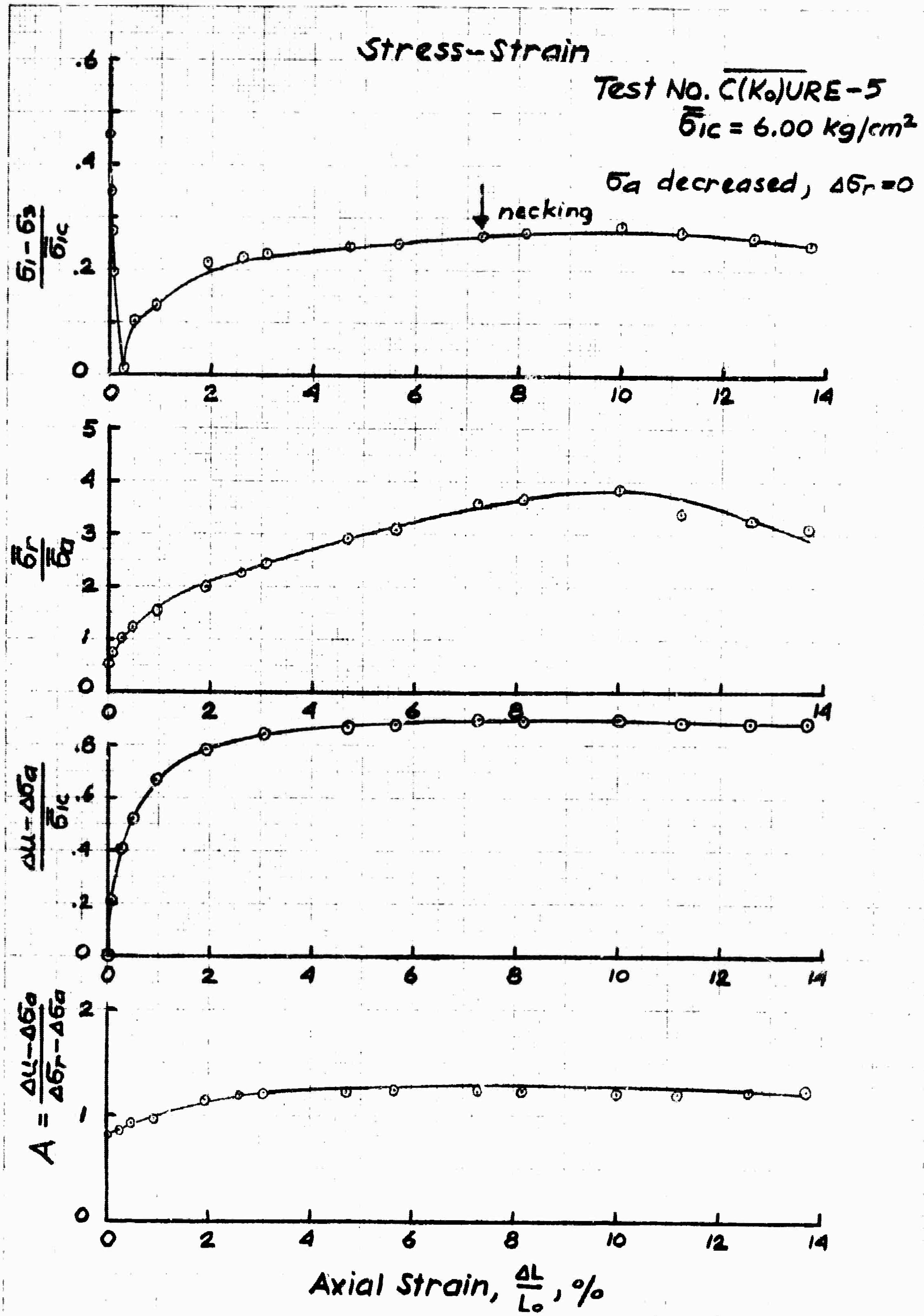


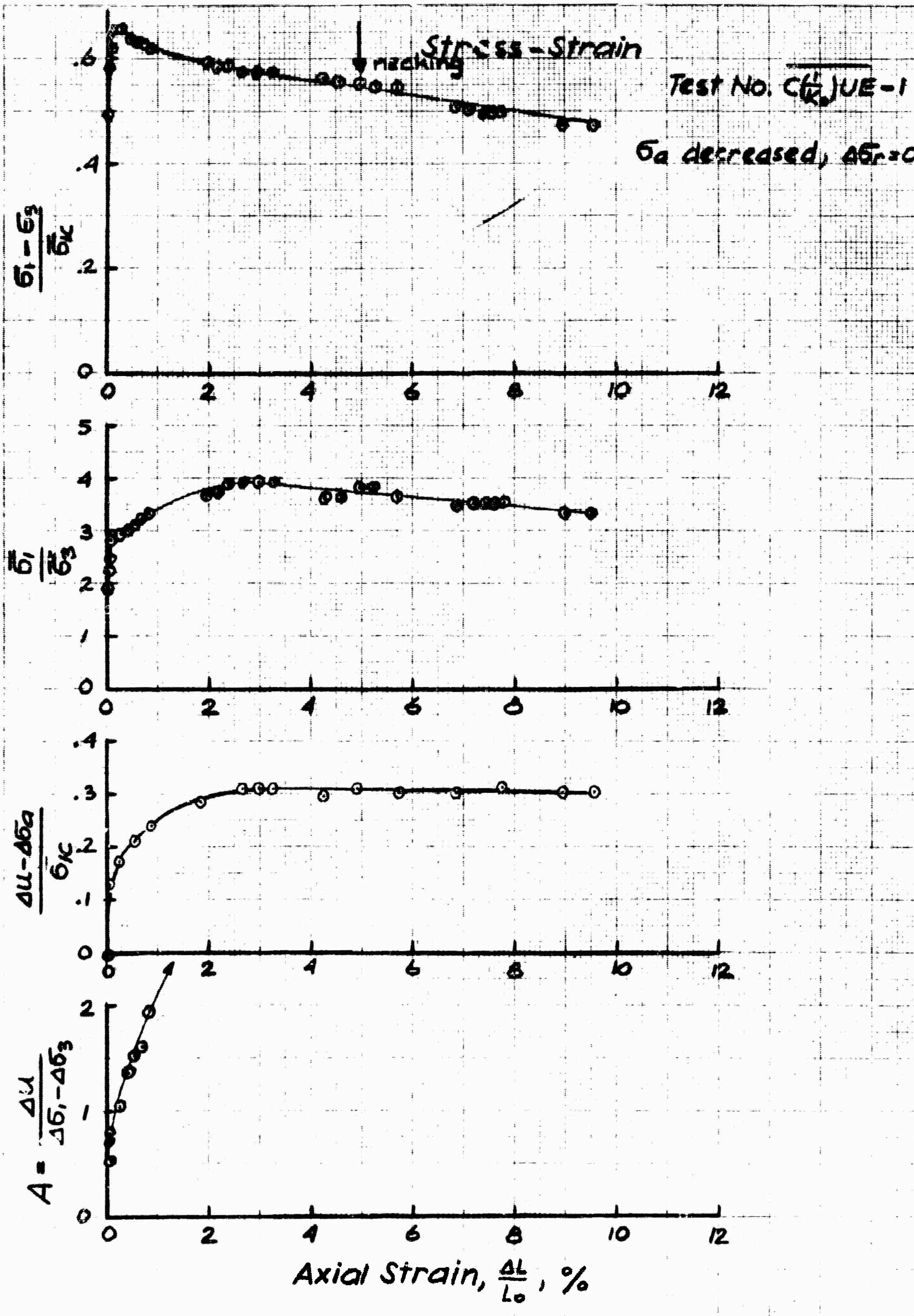
Stress-Strain
Test No. $\overline{C(K_o)}$URE-1
$\bar{\sigma}_{1c}$ = 4.04 kg/cm²
$\bar{\sigma}_r$ increased, $\Delta\bar{\sigma}_a$ = 0
controlled $\bar{\sigma}$
necking
$\frac{\bar{\sigma}_1-\bar{\sigma}_3}{\bar{\sigma}_{1c}}$
$\frac{\bar{\sigma}_r}{\bar{\sigma}_a}$
$\frac{\Delta u-\Delta\bar{\sigma}_a}{\bar{\sigma}_{1c}}$
$A = \frac{\Delta u-\Delta\bar{\sigma}_a}{\Delta\bar{\sigma}_r-\Delta\bar{\sigma}_a}$
Axial Strain, $\frac{\Delta L}{L_o}$, %

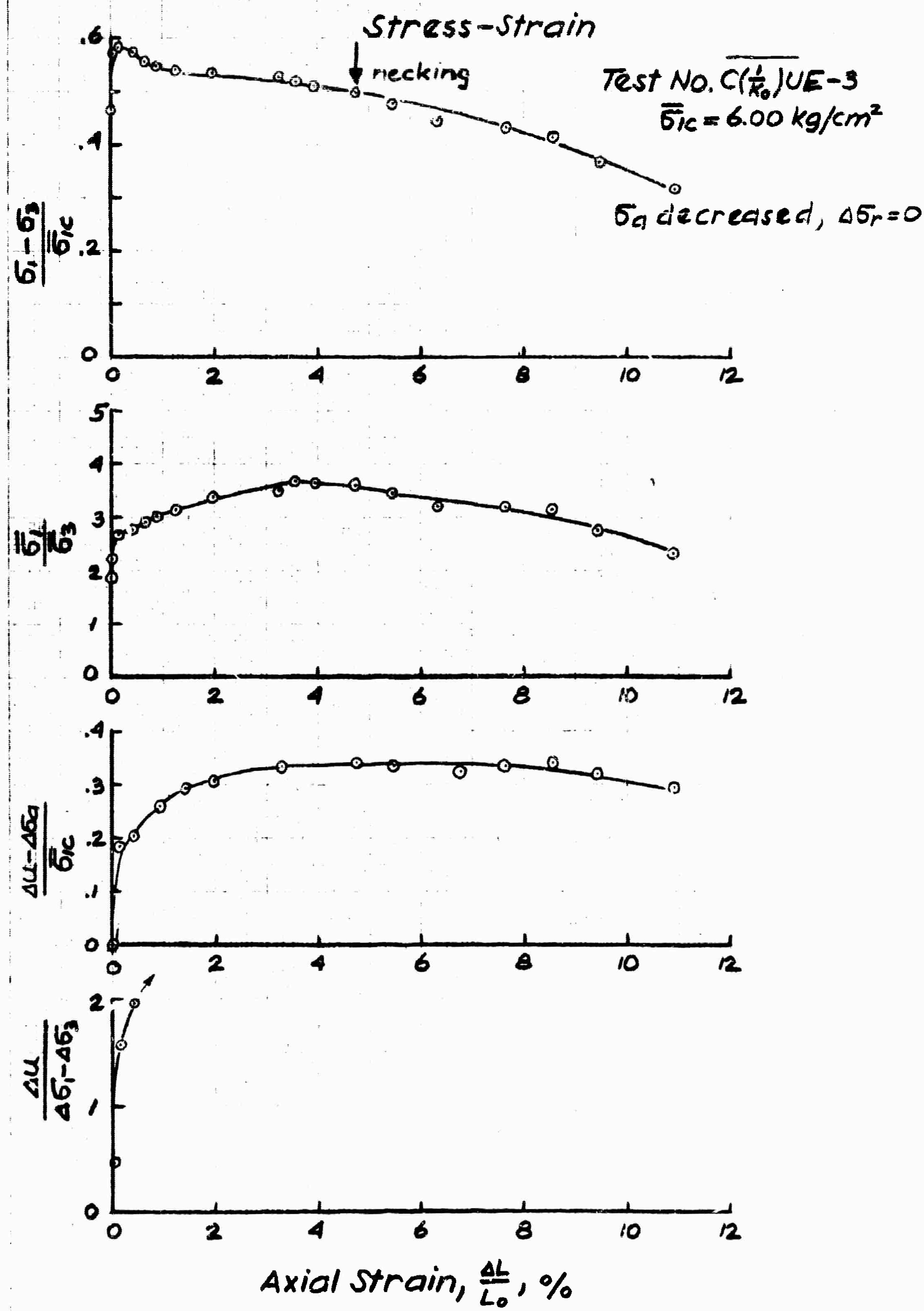


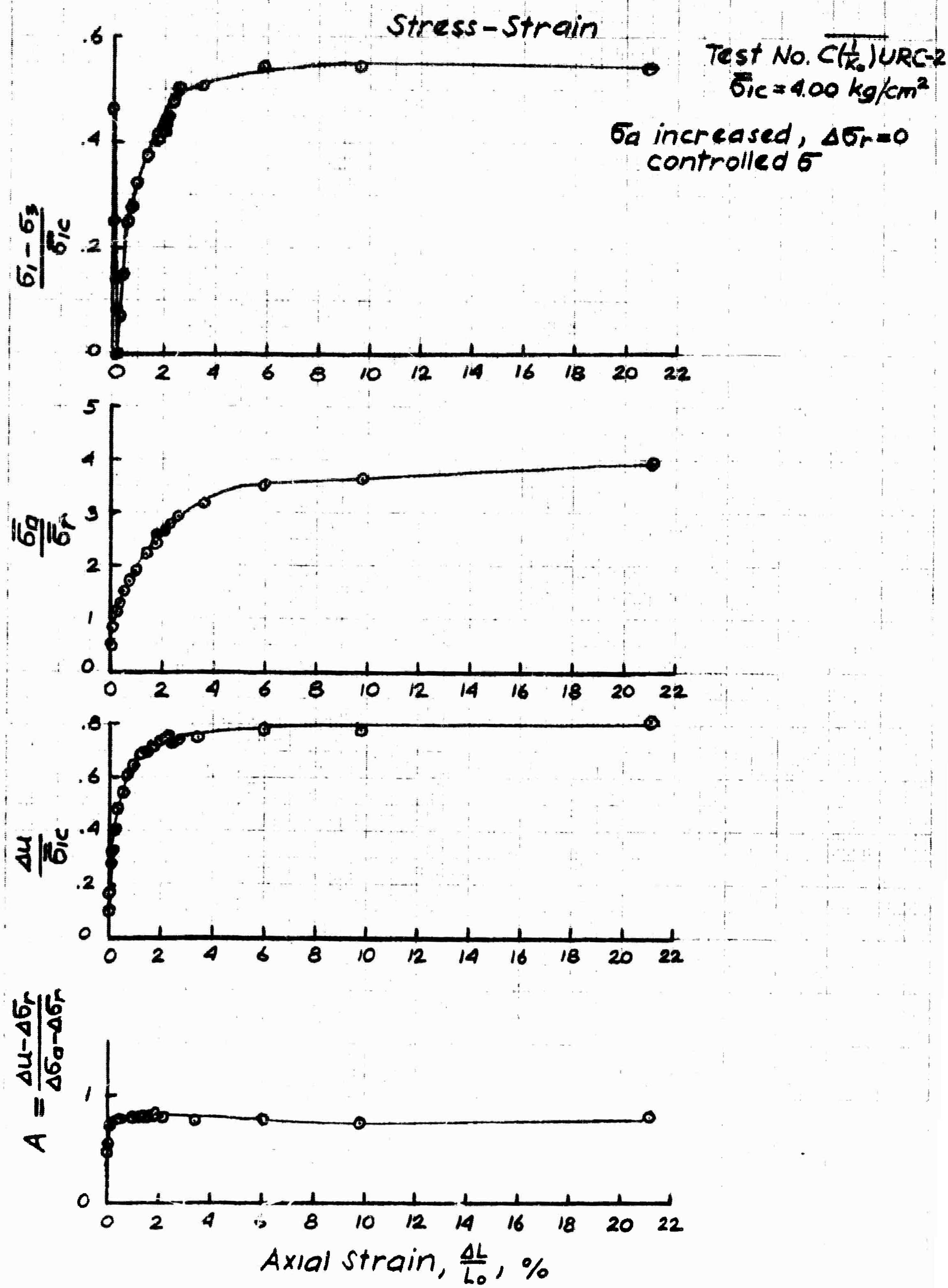
Stress-Strain
Test No. C(K₀)URE-2
σ̄1c = 4.03 kg/cm²
σr increased, Δσa = 0
controlled σ
necking
Axial Strain, ΔL/L₀, %

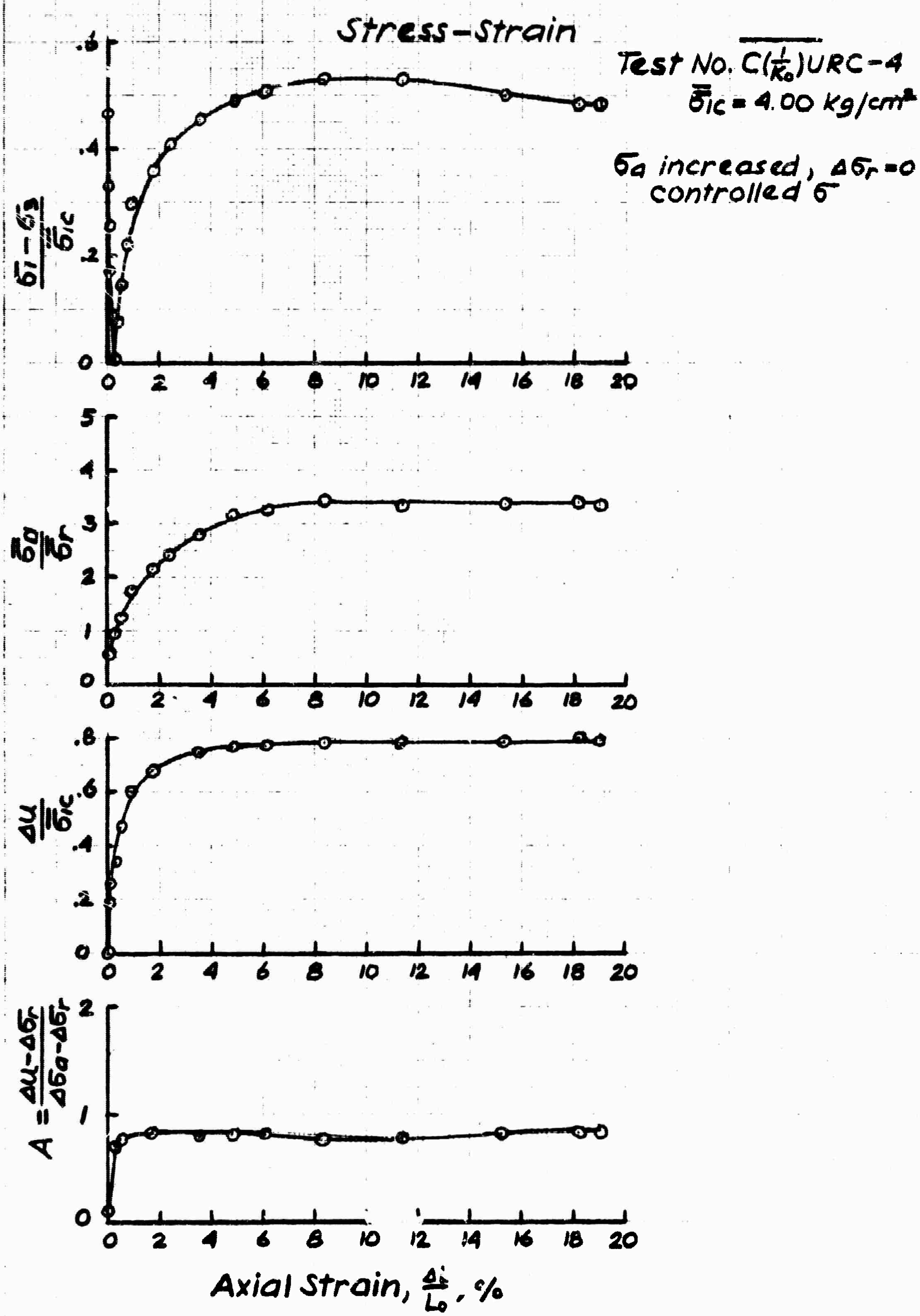

Stress-Strain
Test No. $\overline{C(K_o)}$URE-4
$\bar{\sigma}_{1c}$ = 4.00 kg/cm²
σ_a decreased, $\Delta\bar{\sigma}_r$ = 0
necking
$\frac{\sigma_1-\sigma_3}{\bar{\sigma}_{1c}}$
$\frac{\bar{\sigma}_r}{\bar{\sigma}_a}$
$\frac{\Delta u-\Delta\sigma_a}{\bar{\sigma}_{1c}}$
$A = \frac{\Delta u-\Delta\sigma_a}{\Delta\bar{\sigma}_r-\Delta\sigma_a}$
Axial Strain, $\frac{\Delta L}{L_o}$, %

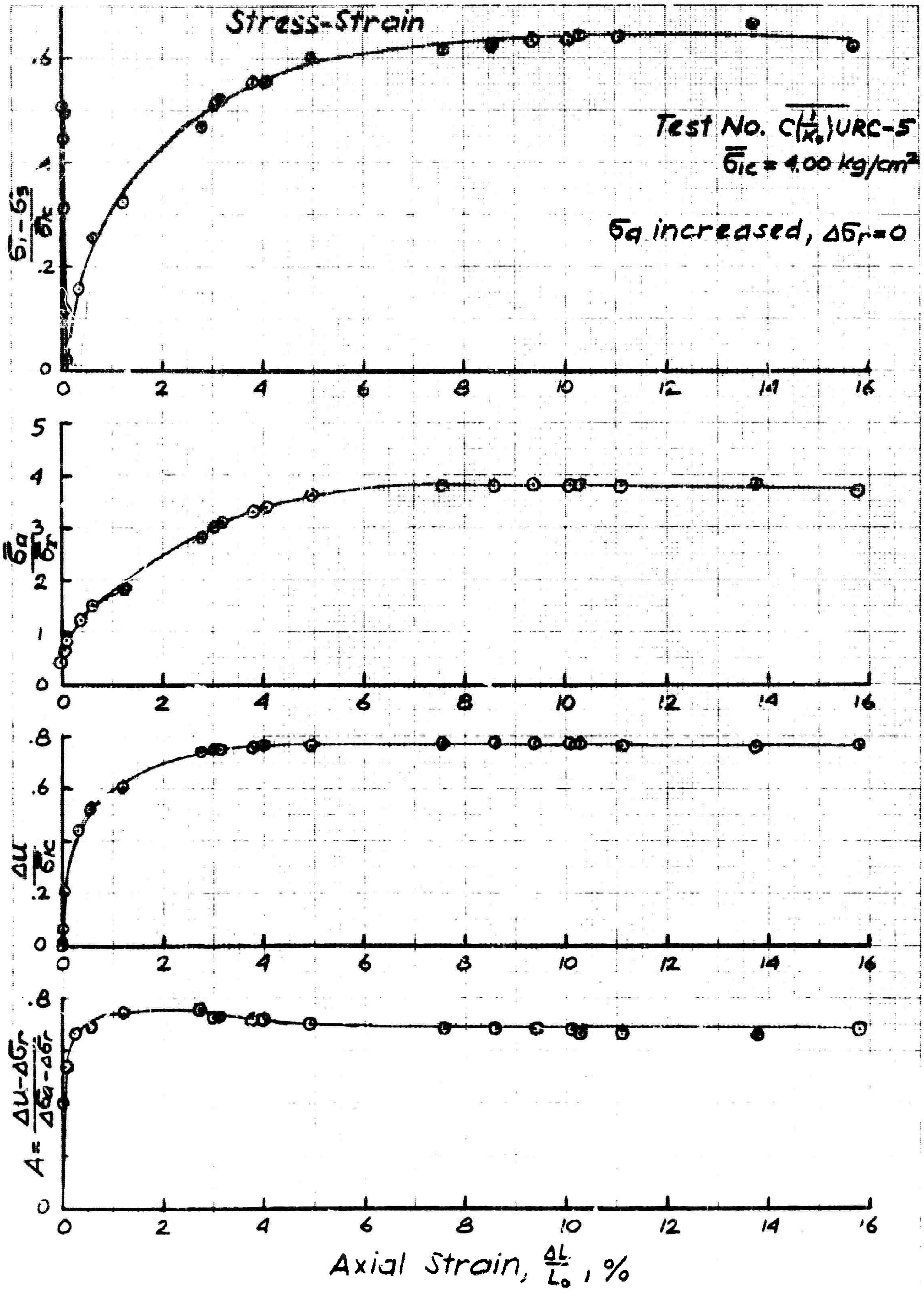

Stress-Strain
Test No. $\overline{C(K_o)URE}$-5
$\bar{\sigma}_{1c}$ = 6.00 kg/cm²
σ_a decreased, $\Delta\sigma_r$ = 0
necking
$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$
$\frac{\bar{\sigma}_r}{\bar{\sigma}_a}$
$\frac{\Delta u - \Delta\sigma_a}{\bar{\sigma}_{1c}}$
$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$
Axial Strain, $\frac{\Delta L}{L_o}$, %


Stress-Strain
necking
Test No. C$(\frac{1}{K_o})$UE-1
σ_a decreased, $\Delta\sigma_r = 0$
$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$
$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$
$\frac{\Delta u - \Delta\sigma_a}{\bar{\sigma}_{1c}}$
$A = \frac{\Delta u}{\Delta\sigma_1 - \Delta\sigma_3}$
Axial Strain, $\frac{\Delta L}{L_o}$, %

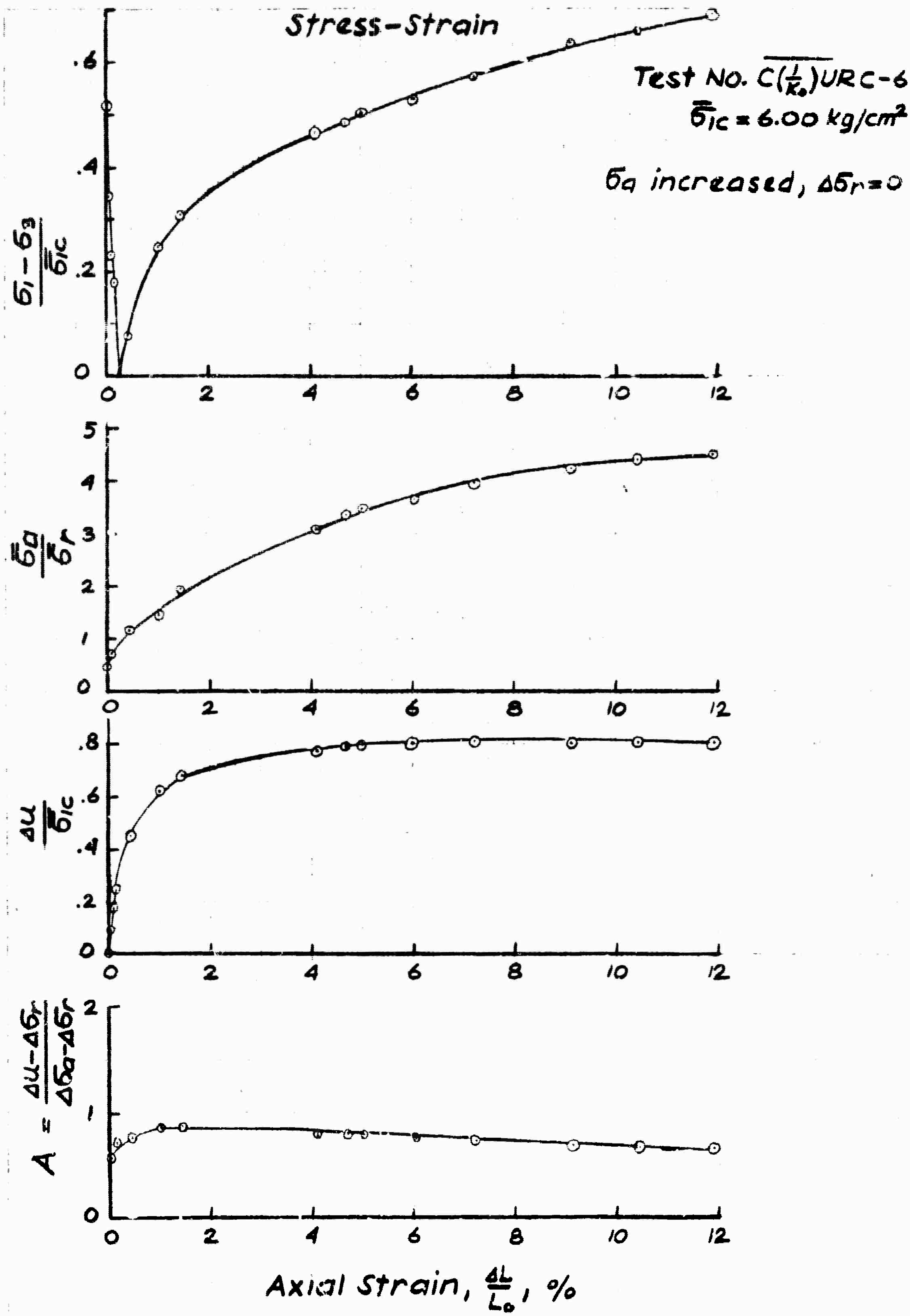

Stress-Strain
necking
Test No. C(1/K₀)UE-3
σ̄1c = 6.00 kg/cm²
σa decreased, Δσr=0
(σ1 - σ3)/σ̄1c
σ̄1/σ̄3
(Δu-Δσa)/σ̄1c
Δu/(Δσ1-Δσ3)
Axial Strain, ΔL/Lo, %


Stress-Strain
Test No. $\overline{C(\frac{1}{K_o})}$URC-2
$\bar{\sigma}_{1c}$ = 4.00 kg/cm²
σ_a increased, $\Delta\sigma_r$ = 0
controlled σ
$\frac{\bar{\sigma}_1-\bar{\sigma}_3}{\bar{\sigma}_{1c}}$
$\frac{\bar{\sigma}_a}{\bar{\sigma}_r}$
$\frac{\Delta u}{\bar{\sigma}_{1c}}$
$A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$
Axial Strain, $\frac{\Delta L}{L_o}$, %


Stress-Strain
Test No. C(1/Ko)URC-4
σ̄1c = 4.00 kg/cm²
σa increased, Δσr = 0
controlled σ
(σ1 - σ3)/σ̄1c
σ̄a/σ̄r
Δu/σ̄1c
A = (Δu-Δσr)/(Δσa-Δσr)
Axial Strain, Δl/Lo, %



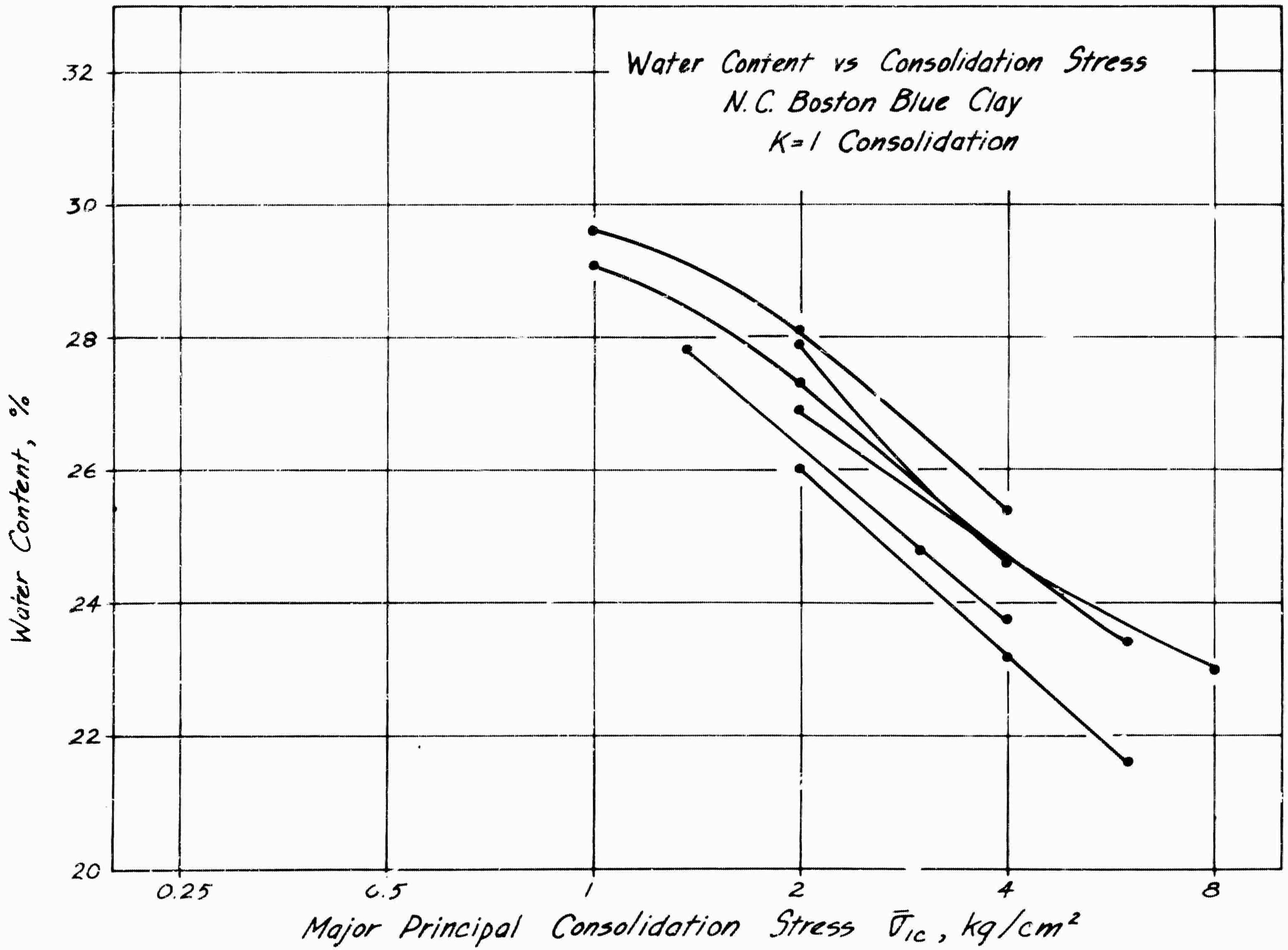
Stress-Strain
Test No. $C(\frac{1}{K_0})$URC-5
$\bar{\sigma}_{1c}$ = 4.00 kg/cm²
σ_a increased, $\Delta\sigma_r$=0
$\frac{\bar{\sigma}_1-\bar{\sigma}_3}{\bar{\sigma}_{1c}}$
$\frac{\bar{\sigma}_a}{\bar{\sigma}_r}$
$\frac{\Delta u}{\bar{\sigma}_{1c}}$
$A=\frac{\Delta u-\Delta\sigma_r}{\Delta\sigma_a-\Delta\sigma_r}$
Axial Strain, $\frac{\Delta L}{L_0}$, %

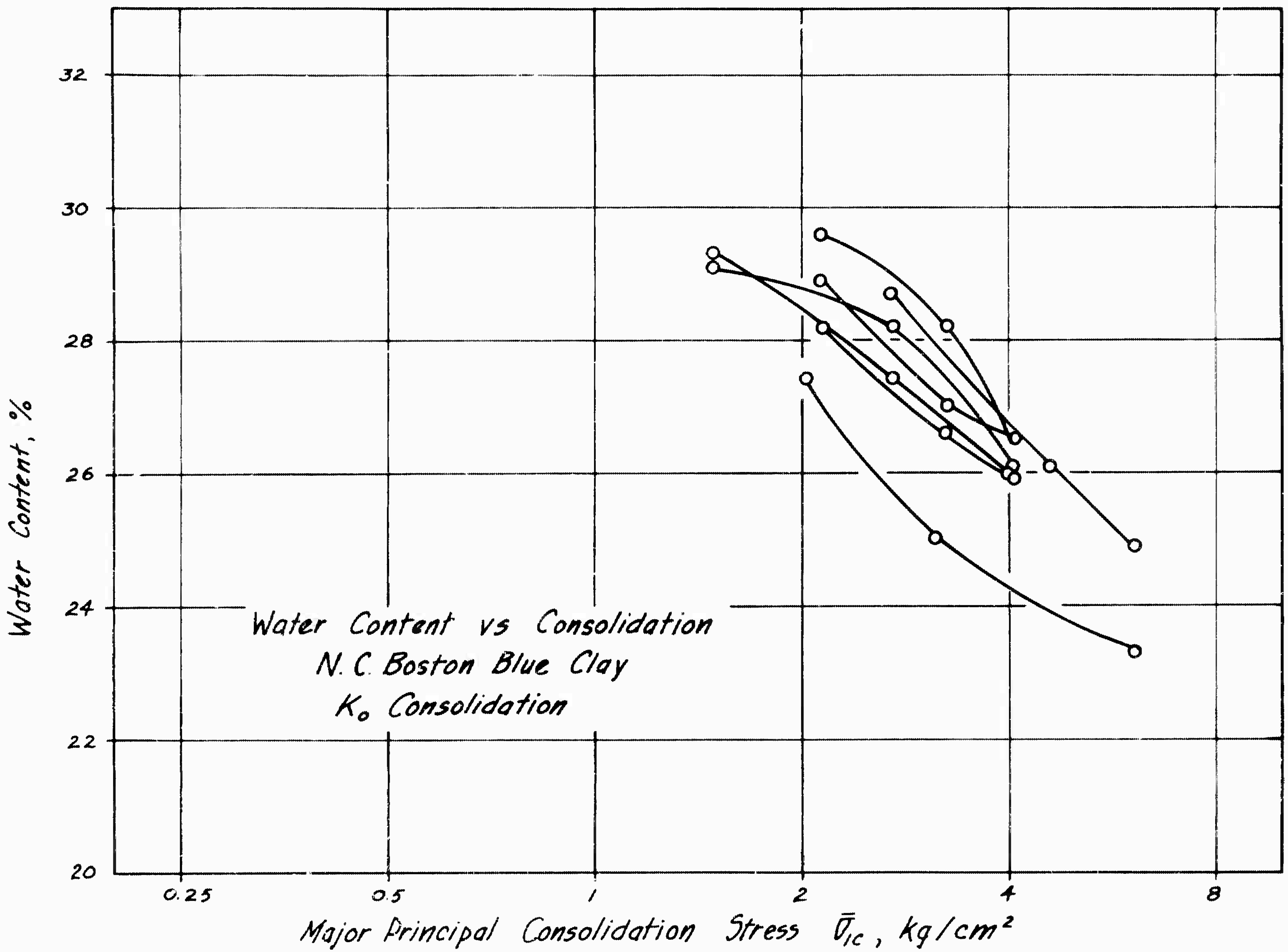


Stress-Strain
Test No. $\overline{C(\frac{1}{K_o})URC-6}$
$\bar{\sigma}_{1c} = 6.00\ kg/cm^2$
σ_a increased, $\Delta\sigma_r = 0$
$\frac{\sigma_1 - \sigma_3}{\bar{\sigma}_{1c}}$
$\frac{\bar{\sigma}_a}{\bar{\sigma}_r}$
$\frac{\Delta u}{\bar{\sigma}_{1c}}$
$A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$
Axial Strain, $\frac{\Delta L}{L_o}$, %

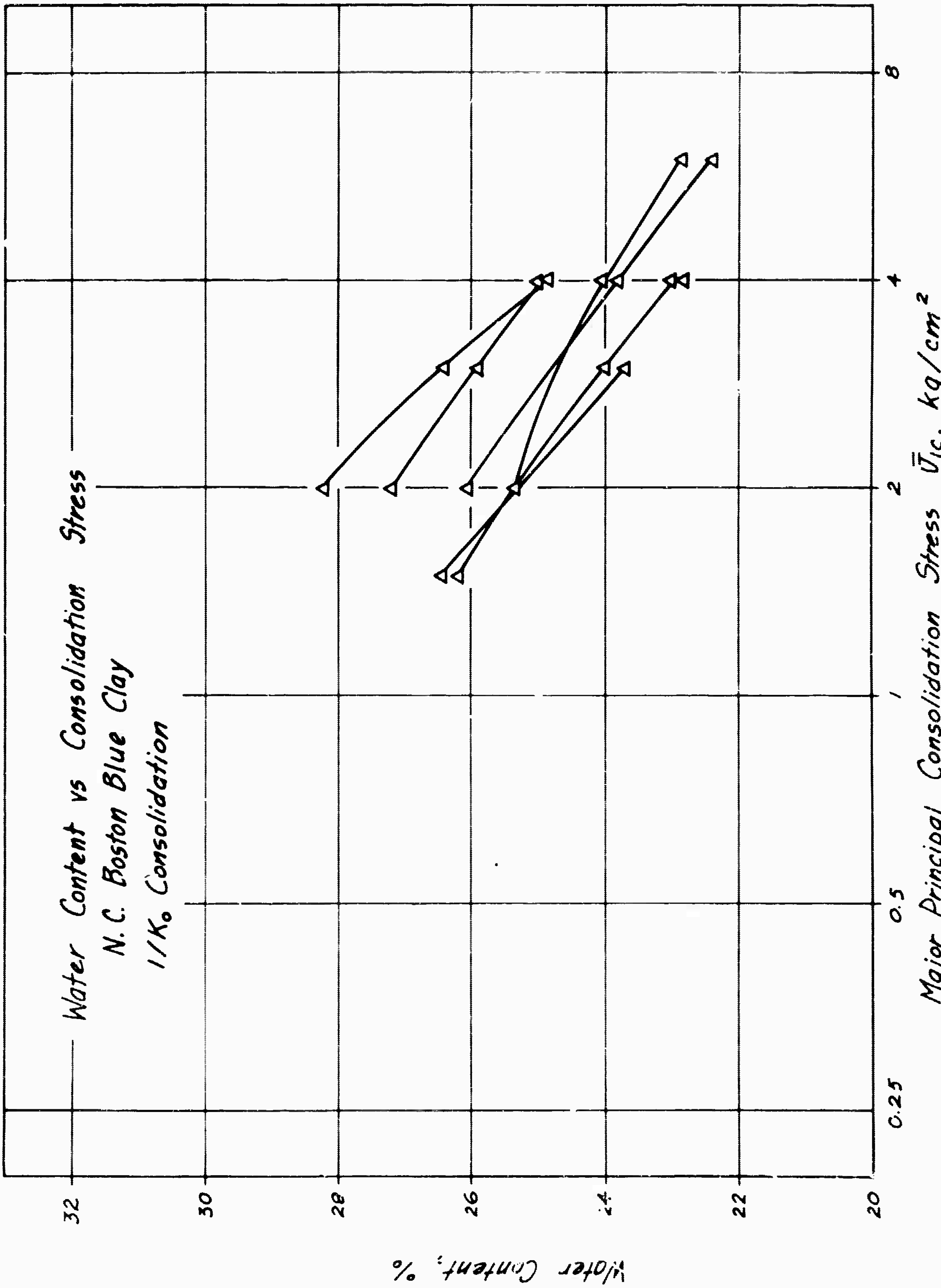
APPENDIX D

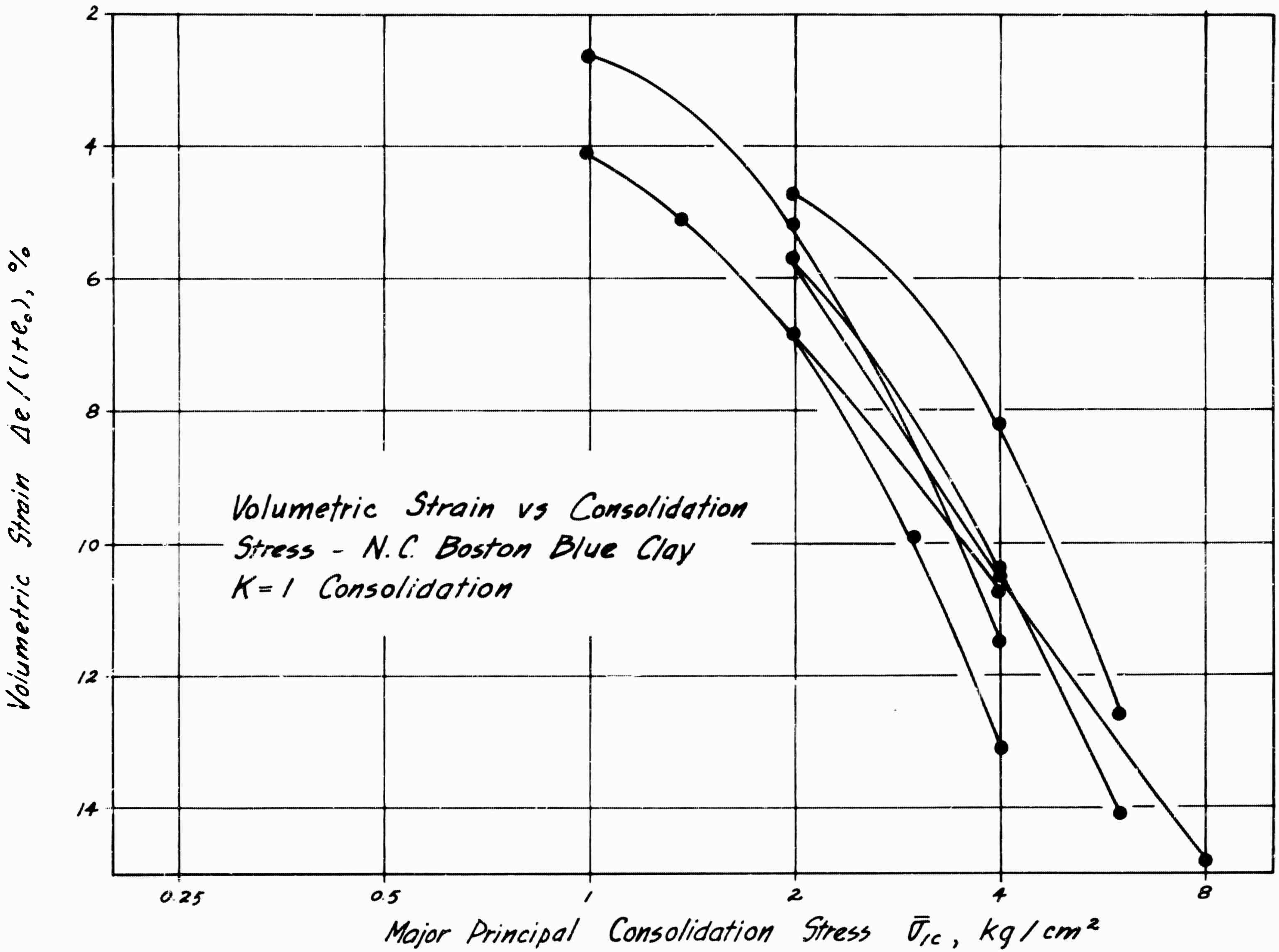
VOLUMETRIC STRAIN AND WATER CONTENT VERSUS MAJOR PRINCIPAL CONSOLIDATION STRESS FROM $\overline{CU}$ TRIAXIAL TESTS ON BOSTON BLUE CLAY


Water Content vs Consolidation Stress
N.C. Boston Blue Clay
K=1 Consolidation
32
30
28
26
24
22
20
0.25
0.5
1
2
4
8
Water Content, %
Major Principal Consolidation Stress $\bar{\sigma}_{1c}$, kg/cm²

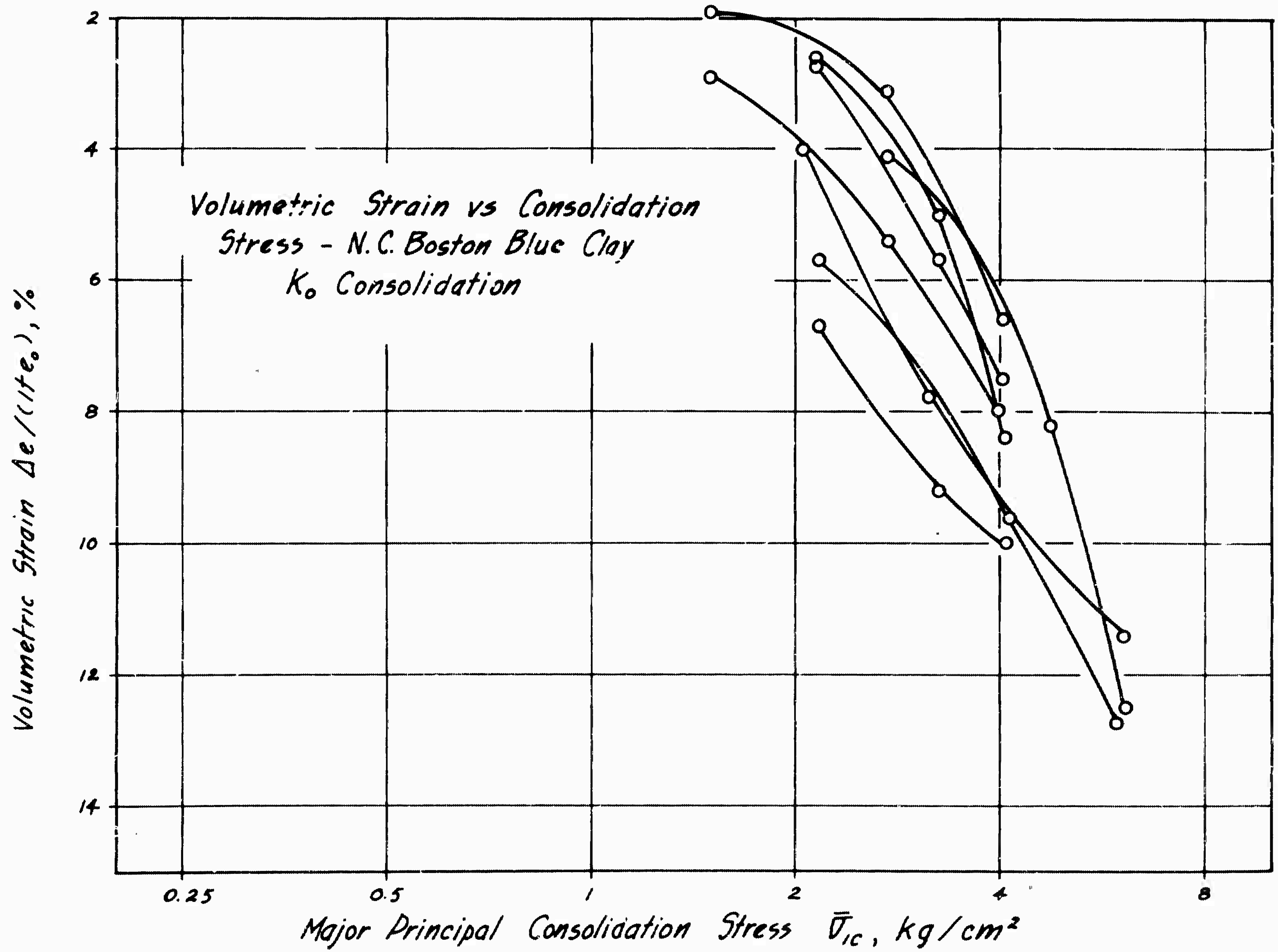


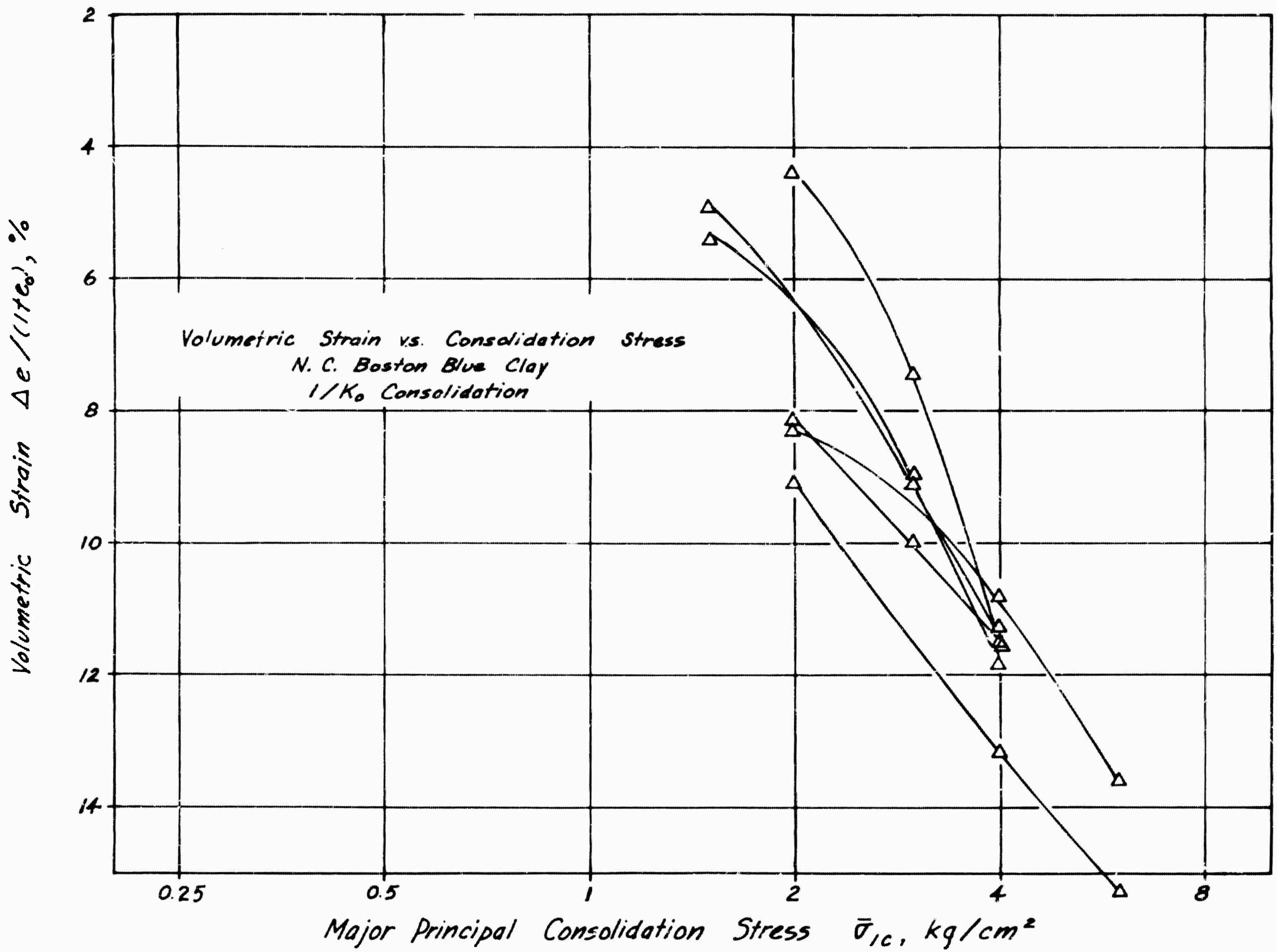
Water Content vs Consolidation
N. C. Boston Blue Clay
K_o Consolidation
Water Content, %
32
30
28
26
24
22
20
0.25
0.5
1
2
4
8
Major Principal Consolidation Stress $\bar{\sigma}_{1c}$, kg/cm²


Water Content vs Consolidation Stress
N.C. Boston Blue Clay
1/K₀ Consolidation
Major Principal Consolidation Stress $\bar{\sigma}_{1c}$, kg/cm²
0.25
0.5
1
2
4
8
Water Content, %
32
30
28
26
24
22
20



Volumetric Strain vs Consolidation
Stress - N.C. Boston Blue Clay
K = 1 Consolidation
Volumetric Strain Δe/(1+e₀), %
2
4
6
8
10
12
14
0.25
0.5
1
2
4
8
Major Principal Consolidation Stress σ̄₁c , kg/cm²


Volumetric Strain vs Consolidation
Stress - N.C. Boston Blue Clay
K_o Consolidation
Volumetric Strain $\Delta e/(1+e_o)$, %
2
4
6
8
10
12
14
0.25
0.5
1
2
4
8
Major Principal Consolidation Stress $\bar{\sigma}_{1c}$, kg/cm²


Volumetric Strain vs. Consolidation Stress
N. C. Boston Blue Clay
1/K₀ Consolidation
Volumetric Strain Δe/(1+e₀), %
2
4
6
8
10
12
14
0.25
0.5
1
2
4
8
Major Principal Consolidation Stress σ̄₁c, kg/cm²